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Nationwide Urban Runoff Program, Tampa, Florida Phase 2 Final Report. Task II.6. Runoff Characterization - Data Analysis. Task II.9. Control Testing - Data Analysis. Task II.10. Projection of Study Area-Wide Loadings Task II.11. Monitor Other NURP Projects

Tampa Dept. of Public Works, FL

Prepared for

Environmental Protection Agency, Washington, DC

Oct 83

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The NURP project tasks reported in this document include (1) Runoff Characterization - Data Analysis, (2) Control Testing - Data Analysis, (3) Projection of Study Area-wide Loadings, and (4) Monitoring Other NURP Projects. These analyses are of stormwater runoff quantity and quality data collected in 5 monitoring basins, and analysis of the effectiveness of 4 types of stormwater controls, including open bottom inlets, drain trenches, ditch systems, and retention/detention ponds. The model input values resulting were used in the determination of stormwater flows and loads for the entire study area.

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CITY OF TAMPA

TAMPA, FLORIDA 33602

TAMPA NATIONWIDE URBAN RUNOFF PROGRAM
PHASE II FINAL REPORT

TASK II.6 RUNOFF CHARACTERIZATION – DATA ANALYSIS
TASK II.9 CONTROL TESTING – DATA ANALYSIS
TASK II.10 PROJECTION OF STUDY AREA-WIDE LOADINGS
TASK II.11 MONITOR OTHER NURP PROJECTS

October, 1983

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October 10, 1983

J-7737

Mr. Ron Giovanelli
City of Tampa
Department of Public Works
City Hall Plaza - 4th Floor
Tampa, FL 33602

Dear Mr. Giovanelli:

We are pleased to submit fifteen copies of the Final Report on Phase II, Tasks II.6, II.9, II.10, and II.11 of the Tampa Nationwide Runoff Program Study. We have incorporated the comments you presented in our August 1983 meeting.

We thank you for your cooperation and for the assistance and information which you have provided to us during this work.

Very truly yours,

Abu M.Z. Alam

Abu M.Z. Alam
Associate

AMZA/jln

New York / Palo Alto / San Bernardino / Arlington Heights, IL / Chicago / Houston / Atlanta / Somerville, NJ / Silver Spring / Honolulu

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CHAPTER 1

INTRODUCTION

Background

The City of Tampa, Florida is experiencing rapid growth and urbanization, and associated flooding problems. Pollution of the lower Hillsborough River estuary, which runs through the center of the City, is partially attributable to stormwater runoff. A program designed to effectively address flooding and pollution concerns in the Lower Hillsborough River Basin must incorporate State of Florida Department of Environmental Regulation criteria in stormwater permitting and water quality.

The implications of such an integrated approach to stormwater quantity and quality issues consistent with regulation has not been investigated. For this reason, the City of Tampa has participated in the Nationwide Urban Runoff Program (NURP). NURP is a program designed to determine the characteristics of stormwater runoff and extent of runoff quantity and quality problems on a nationwide basis. Measurements and analysis are being conducted at twenty-eight cities of which Tampa is one.

The Tampa NURP study consists of three phases as follows:

I. Assessment of stormwater loads and their receiving water impacts based on existing data.

II. Data collection and analysis of rainfall, stormwater runoff, and stormwater controls.

III. Development of a Stormwater Management Plan for the
City of Tampa.

A related study on the impact of stormwater on the lower Hillsborough River is being sponsored by the City of Tampa.

Phase I of the Tampa NURP Study has been completed. Major goals of the remainder of the Tampa NURP Study are to characterize the stormwater flows and loads from urban drainage basins, analyze the effectiveness of selected stormwater controls, determine the impact of storm generated loads on the lower Hillsborough River, and develop an overall Stormwater Management Plan (SMP) for the City of Tampa. The SMP will address existing receiving water quality problems due to the quantity and quality aspects of stormwater runoff, support the City's efforts to deal with flooding problems in an environmentally sound manner, and satisfy State requirements for stormwater permits.

Report Scope and Organization

This report contains Tasks II.6, II.9 and II.10 of Phase II of the Tampa NURP Study. A list of Tasks and their references are given in Table 1-1 for the entire Phase II Study.

This report, along with the Task II.2 Rainfall Quality Analysis report (3), involve the analysis of the collected rainfall-runoff data. The other Phase II Tasks comprise the efforts related to site selection and collection of stormwater runoff quantity and quality and rainfall quantity and quality data (1, 3, 4, 5, 6, 7, 8, 9).

TABLE 1-1. SUMMARY OF TAMPA NURP PHASE II TASKS

Task No.	Task name	Reference
II.1	Procure and Install Gaging and Sampling Equipment	(1)
II.2	Rainfall Quantity Analysis	(2)
II.3	Rainfall Quality Analysis	(3)
II.4	Runoff Characterization - Basin Selection and Equipment Design	(4)
II.5	Runoff Characterization - Sampling	(5, 6)
II.6	Runoff Characterization - Analysis	(this report)
II.7	Control Testing - Basin Selection and Equipment Design	(7)
II.8	Control Testing - Sampling	(8, 6)
II.9	Control Testing - Analysis	(this report)
II.10	Projection of Study Area-wide Loadings	(this report)
II.11	Monitor Other NURP Projects	(this report)
II.12	Quality Assurance Program	(9)

Task II.6 involves the analysis of stormwater runoff quantity and quality data collected in five monitoring basins. The analysis is performed using both single event stormwater quantity and quality simulations and statistical analyses. The Task II.6 analysis is presented in Chapter 2.

Task II.9 involves the analysis of the effectiveness of four types of stormwater controls, including open bottom inlets, drain trenches, ditch systems, and retention/detention ponds. The first three controls were analyzed for hydraulic effectiveness, and the retention/detention ponds were analyzed for both flow and pollutant removal capabilities. This work is presented in Chapter 3.

Task II.10 is the development of stormwater flows and loads for the entire study area, the lower Hillsborough River drainage basin. These flows and loads are developed using continuous rainfall-runoff simulation for long-term rainfall records, based on model input values developed in Tasks II.6 and II.9. This analysis is presented in Chapter 4.

The information presented in this Report will be used as a basis for development of the Stormwater Management Plan for the City of Tampa during Phase III and as input to the assessment of impacts of stormwater on the lower Hillsborough River during the Receiving Water Study.

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4. Priede-Sedgwick, Inc., Runoff Characterization and Watershed Selection, Task II.4, Tampa Nationwide Urban Runoff Program Study, Phase II, December, 1982.

5. Priede-Sedgwick, Inc., Runoff Characterization Water Quality and Flow Data, Task II.5, Tampa Nationwide Urban Runoff Program Study, Phase II, March, 1983.

6. Priede-Sedgwick, Inc., Detailed Event Data, Tasks II.5 and II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, April, 1983.

7. Priede-Sedgwick, Inc., Control Testing Basin Selection, Task II.7, Tampa Nationwide Urban Runoff Program Study, Phase II, June, 1982.

8. Priede-Sedgwick, Inc., Control Testing Water Quality and Flow Data, Task II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, March, 1983.

9. Priede-Sedgwick, Inc., Quality Assurance/Quality Control Procedures Manual, Task II.12, Tampa Nationwide Urban Runoff Program Study, Phase II, December, 1982.

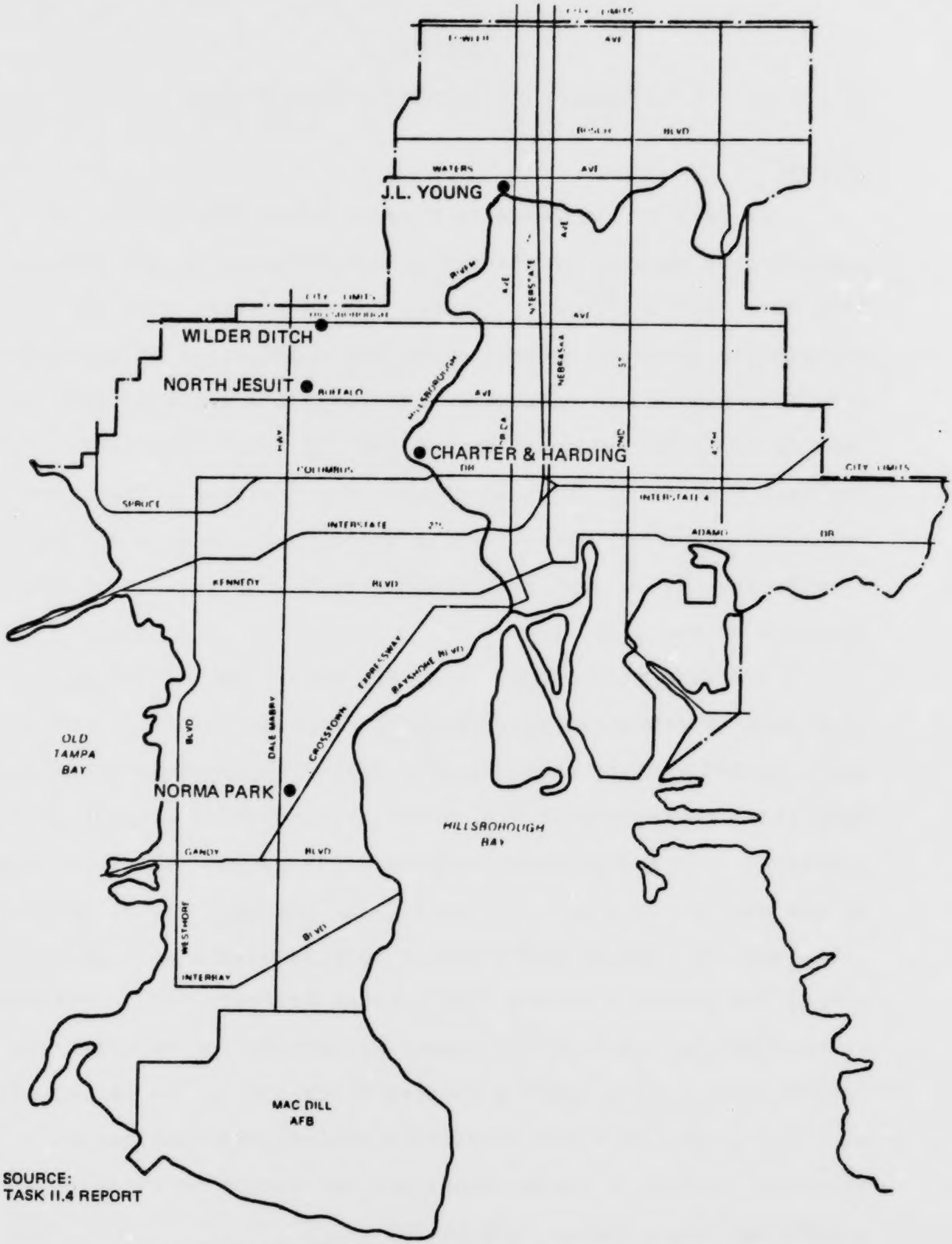
CHAPTER 2

ANALYSIS OF STORMWATER RUNOFF DATA

General

As part of the Phase II City of Tampa NURP Study, considerable amounts of stormwater runoff quantity and quality data have been collected. In this Chapter, these data are analyzed in order to determine the characteristics of stormwater runoff in Tampa, and determine what factors are significant in causing the observed quantity and quality characteristics. This analysis will be utilized in Chapter 4 to develop a basis for projection of study area-wide flows and loads, and to assess the transferability of data from monitoring basins for use in other portions of the study area.

The data used in this analysis were obtained during a stormwater measurement program which recorded rainfall, runoff quantity and quality characteristics at five drainage basins. Detailed descriptions of the basins, the sampling program and collected data are provided elsewhere (1, 2, 3). The locations of the basins are shown in Figure 2-1. For each basin, about three discrete events and twelve composite events were sampled during the period from May 1982 through February 1983. Discrete event sampling involved continuous measurement of rainfall and runoff with several quality samples taken during the period of runoff. Composite event sampling also involved continuous rainfall and runoff measurement, but one composite water quality sample was taken during each storm.


CITY LIMITS
FOWLER AVE
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J.L. YOUNG
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AVE
INTERSTATE
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WAY
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COLUMBUS DR
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INTERSTATE 4
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BAYSHORE BLVD
OLD TAMPA BAY
BLVD
DALE MABRY
NORMA PARK
HILLSBOROUGH BAY
GANDY BLVD
WESTSHORE
BLVD
INTERBAY
MAC DILL AFB

SOURCE:
TASK II.4 REPORT

FIG. 2-1 SITE LOCATION OF RUNOFF CHARACTERIZATION BASINS

Analysis Approach

In characterizing the stormwater runoff, it was important to analyze and develop an understanding of the significant components in the rainfall-runoff process. The discrete event stormwater sampling program was conducted to monitor within-storm variations of rainfall, and stormwater quantity and quality for representative drainage basins in Tampa. By analysis of this within-storm variability, an understanding of the rainfall-runoff process was developed. Because of the highly variable nature of stormwater quality, it is sometimes necessary to analyze quality data from a large number of storm events, such as in the composite event sampling program. Composite sampling of a large number of storm events can be used to determine the range of stormwater quality characteristics expected.

The information obtained through the field program reflects only the specific circumstances and site characteristics where measurements were made. It is equally important to be able to predict the operation of the stormwater system under a variety of possible stormwater flow and load conditions. Therefore, stormwater modeling of the Tampa system was conducted.

The prediction of how the system operates under both existing and future conditions was accomplished by using two mathematical models: The Storm Water Management Model (SWMM) (4), as modified by Metcalf & Eddy, Inc., and the Simplified Storm Water Management Model (S/SWMM) (5).

SWMM represents the complex time-varying physical processes during which pollutants are accumulated on the surface, washed off during storm events, and transported through the storm sewer system. These processes were simulated to characterize flows from single storm events. In addition to a detailed analysis of single events, it is important to understand the long-term (seasonal and annual) operational characteristics of the existing system or of any proposed modification. In this case the information of interest includes the number, frequency, and quantity of flood events as well as the quality and quantity of runoff to the River. S/SWMM was used to generate this information. S/SWMM schematizes the runoff-transport processes to several simple steps so that long-term simulation becomes economically feasible. The differences between the two models are summarized in Table 2-1. SWMM is described in detail in this chapter and S/SWMM is described in Chapter 4.

Analysis of Discrete Event Data

SWMM was the primary tool used to analyze the discrete data and describe within-storm variability of stormwater quantity and quality. In this section, the SWMM and model inputs are first defined, and then the process of calibration and verification is summarized. Finally, conclusions are drawn regarding the processes identified through the analysis which appear to be significant in causing the observed stormwater characteristics in Tampa.

TABLE 2-1. CHARACTERISTICS OF SWMM AND S/SWMM

Characteristic	SWMM	S/SWMM
Simulation period	Single storm (generally less than 24 hours)	Seasonal, annual
Time step	10 - 15 minutes	1-24 hours (normally one hour)
Basis of runoff calculations for a given drainage area	Mathematical description of physical processes such as infiltration, surface storage etc. on both pervious and impervious areas	Single runoff coefficient used to convert rainfall into runoff
Size of typical drainage area	10 - 200 acres	100 - 1,000 acres
Basis for transporting flows and pollutants	Flow routing through all components in the collection system (i.e., pipes and manholes)	Allowing volumes of runoff to flood after subtracting available interceptor capacity and storage capacity
Quantification of pollutants for a given drainage area	Variable concentrations based on deposition in pipes and surface wash-off	Constant concentrations
Calibration/verification	Accomplished at various locations throughout the system	Possible only at measured overflow locations

Description of SWMM and Model Inputs. SWMM is a comprehensive, deterministic model which simulates stormwater runoff quantity and quality. The basic input which drives SWMM is a rainfall event, described by time-varying rainfall intensities. Once the rainfall event is selected, the physical processes which route the flow and quality through the drainage system are divided into two "Blocks": the Runoff Block, which simulates surface runoff processes; and the Transport Block, which simulates collection system processes, and produces results in the form of time-varying quantity (hydrographs) and quality (pollutographs) values. These Blocks and the Executive Block which controls their use are described here. Figure 2-2 illustrates a schematic of a separate storm sewer system typical of Tampa, and Figure 2-3 illustrates the major components of SWMM.

Executive Block - The Executive Block is the main control block. It has the following functions:

- Assigns logical file numbers (tape/disk/drum).
- Maintains control of execution of all other blocks.
- Produces user-selected results such as plots.

Runoff Block - Each of the five monitored drainage basins were divided into smaller subcatchments (0.5 to 40 acres) based on the existing drainage patterns. The runoff and pollutants from each subcatchment were defined for input into SWMM. This process included defining each subcatchment in terms of length,

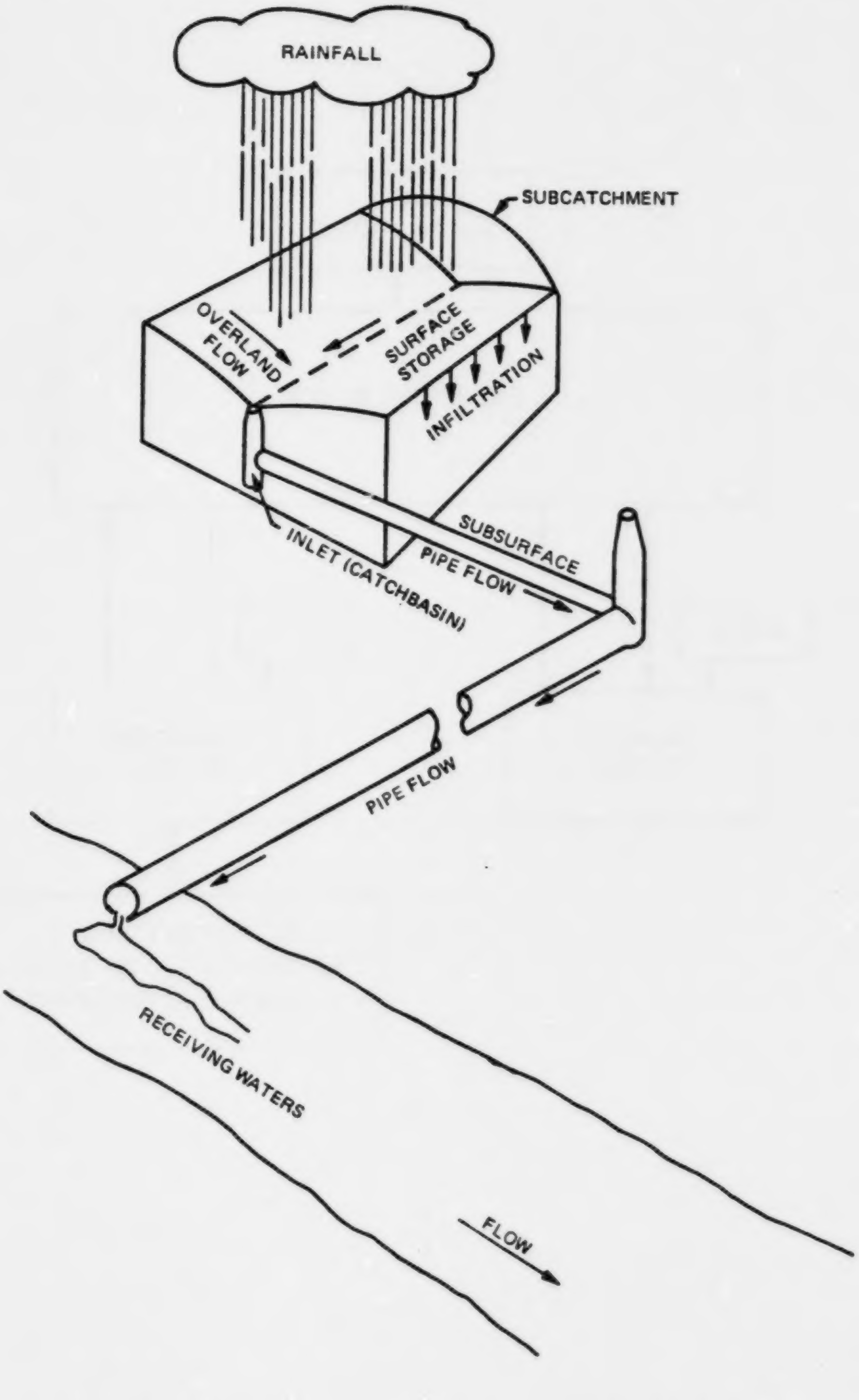

RAINFALL
SUBCATCHMENT
OVERLAND FLOW
SURFACE STORAGE
INFILTRATION
INLET (CATCHBASIN)
SUBSURFACE PIPE FLOW
PIPE FLOW
RECEIVING WATERS
FLOW

FIG. 2-2 SCHEMATIC DIAGRAM OF STORM SEWER SYSTEM AS REPRESENTED IN SWMM

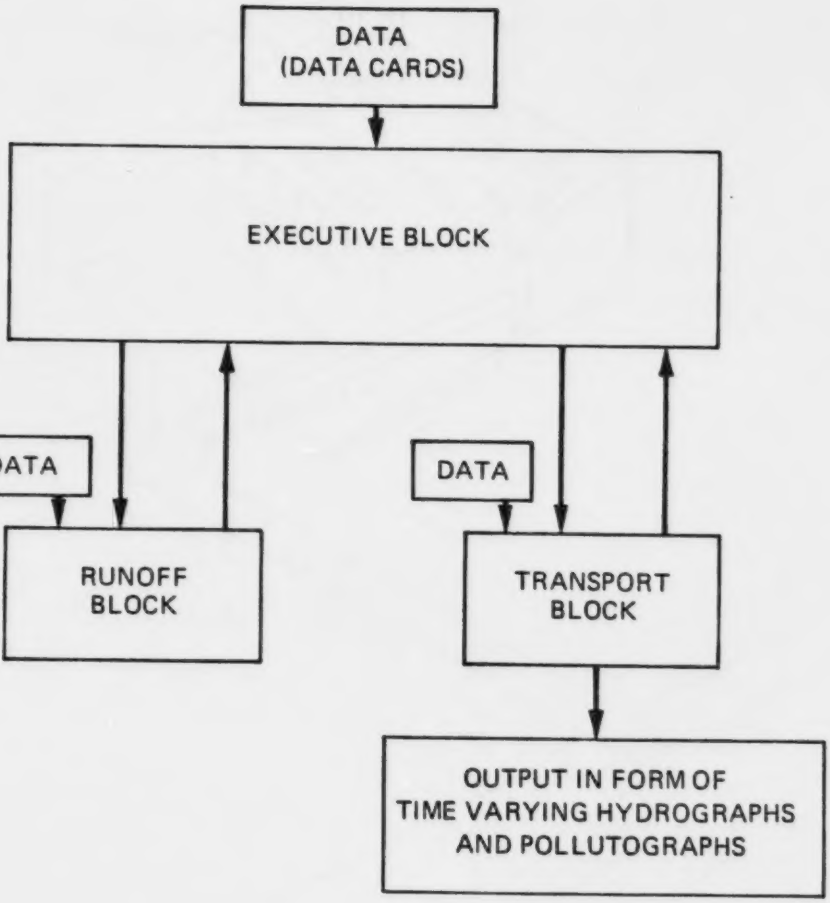

DATA
(DATA CARDS)
EXECUTIVE BLOCK
DATA
RUNOFF
BLOCK
DATA
TRANSPORT
BLOCK
OUTPUT IN FORM OF
TIME VARYING HYDROGRAPHS
AND POLLUTOGRAPHS

FIG. 2-3 MAJOR COMPONENTS OF STORM WATER MANAGEMENT MODEL

width and total area, percent impervious area, ground slope, surface resistance factors, surface storage factors and infiltration rates. After the subcatchments were defined, they were divided into the separate land uses which exist within their boundaries. SWMM allows input of the following five land uses:

- single-family residential,
- multi-family residential,
- commercial,
- industrial, and
- open space.

The street sweeping frequency, number of sweeping passes, number of dry weather days before a storm, and surface pollutant accumulation rates were also input to SWMM. These were used in conjunction with land use and gutter length to compute the amount of surface pollutants accumulated before any particular storm event.

It was judged that for the short duration, high intensity storms identified in Task 2.2 (6) as the storms most likely to wash off significant pollutant loadings, runoff from pervious areas (grasses and open space) would be small. For this reason, the infiltration rate was initially set high and was simulated in SWMM using Horton's equation. The initial rate was 3.0 inches per hour decreasing rapidly to a minimum of 0.5 inches per hour. The surface storage (ponding) on pervious areas was initially set at 0.25 inches. These two factors were adjusted in each basin, if necessary, in the calibration of runoff peaks and volumes.

Surface storage on 75 percent of the impervious areas (roofs and pavements) was initially judged to be 0.06 inches. No surface storage was allowed on the remaining 25 percent. This parameter was also considered a candidate for adjustment during the calibration process. Parameters such as percent impervious, ground slope and land use, were based on the data on basin characteristics collected during Phase II (1), and are generally not expected to be adjusted during the calibration process.

At the beginning of the rainfall event, the portion of the rain which becomes runoff carries surface pollutants into the collection system to be transported downstream. Once the program has calculated the quantity and quality of surface runoff, it proceeds to the Transport Block.

Transport Block - The Transport Block describes the wet weather flows in the stormwater drainage system and the pollutants carried by the flow. The major input data consists of the physical description of the collection system including the size, slope and roughness coefficients for storm sewers and ditches. Details of the drainage system were also determined from previous Phase II outputs (1). Generally, pipes and ditches 12 inches and larger were modeled.

The following hydraulic parameters were used in the Storm Water Management Model:

Suface retention storage

a. impervious areas: 1/16 to 3/8 inch

b. pervious areas: 1/4 to 3.0 inch

Ground infiltration, as used in Horton's Equation

a. maximum rate: 3.0 inch/hour

b. minimum rate: 0.5 inch/hour

c. exponential decay rate: 0.00115 sec.^{-1}

Flow friction factors (n) as used in Manning's Equation

a. Overland flow

- impervious surfaces: 0.013
- pervious surfaces 0.25

b. concrete sewers: 0.013

c. ditches: 0.022

Additional input includes any base flow or estimated groundwater infiltration entering the collection system during a particular storm event. Using the physical characteristics of the sewer system described above, the Transport Block routes the stormwater quantities computed by the Runoff Block through the collection system and produces output hydrographs and pollutographs which describe the variation in outflow quantity and quality over time. Although the characteristics of the stormwater are normally computed for the most downstream element in a particular basin, the Transport Block will produce hydrographs and pollutographs for intermediate points in the collection system, if desired, to analyze problem areas and verify the accuracy of modeling the collection system.

Flow, five day biochemical oxygen demand (BOD_5), suspended solids (SS) and total coliform bacteria were simulated in the Runoff and Transport Blocks. These water quality parameters have been selected because coliform has been a problem in the lower Hillsborough River specifically attributed to stormwater (7); dissolved oxygen is also an identified problem in the River. thus requiring quantification of BOD loads; and BOD and coliform quantities in stormwater are computed by SWMM as a function of solids accumulation and wash-off in each basin.

SWMM Calibration and Verification - Hydraulics. In order to identify important factors in the rainfall-runoff process in Tampa, and allow projection of loadings under design conditions, the SWMM model was calibrated and verified for each of the five basins monitored. The calibration and verification process for hydraulics is described in this section.

J.L. Young Basin - This basin has a land area of 8.8 acres and consists of an apartment complex. Due to the parking areas and roofs, approximately 60 percent of the basin is impervious with roof leaders and inlets connecting to the storm drainage system. The storm sewers are concrete with diameters ranging from 12 to 30 inches. The rain gage for this basin was located near the outflow monitoring station.

In this basin the calibration and verification of SWMM was conducted using the discrete storm event data obtained on August 26, September 10, and September 21, 1982. The characteristics of these storms are summarized in Table 2-2, and the comparisons of measured and computed stormwater flows are presented in Figure 2-4. As shown, the outflow hydrographs computed by SWMM compare favorably with the observed hydrographs.

TABLE 2-2. J.L. YOUNG - STORM CHARACTERISTICS FOR HYDRAULIC CALIBRATION

Storm date	Total rain, in.	Duration, hr.	Dry weather before storm, days	Intensity, in/hr Peak	Intensity, in/hr Average	Base flow, cfs.
August 26, 1982	0.80	1.63	1	1.52	0.49	0.03
September 10, 1982	0.20	1.33	2	0.48	0.15	0.02
September 21, 1982	0.24	4.25	0	0.32	0.06	0.56

Discrepancies between the computed and observed hydrographs may be largely attributed to discrepancies in the observed data. These can be summarized as follows:

1. Storm of September 10 - a large increase in flow at about 14:45 hours which could not have occurred based on the rainfall input.
2. Storm of September 10 and 21 - a delay of about 10 minutes between the simulated and the observed start of runoff which can not occur given the specified rainfall input.

An underlying problem with rainfall-runoff measurement in Tampa has been that rainfall exhibits significant spatial variability. This large areal variability occurs especially during summer thunderstorms. It is thus possible, for this basin as well as others, that the rainfall which affects most of the

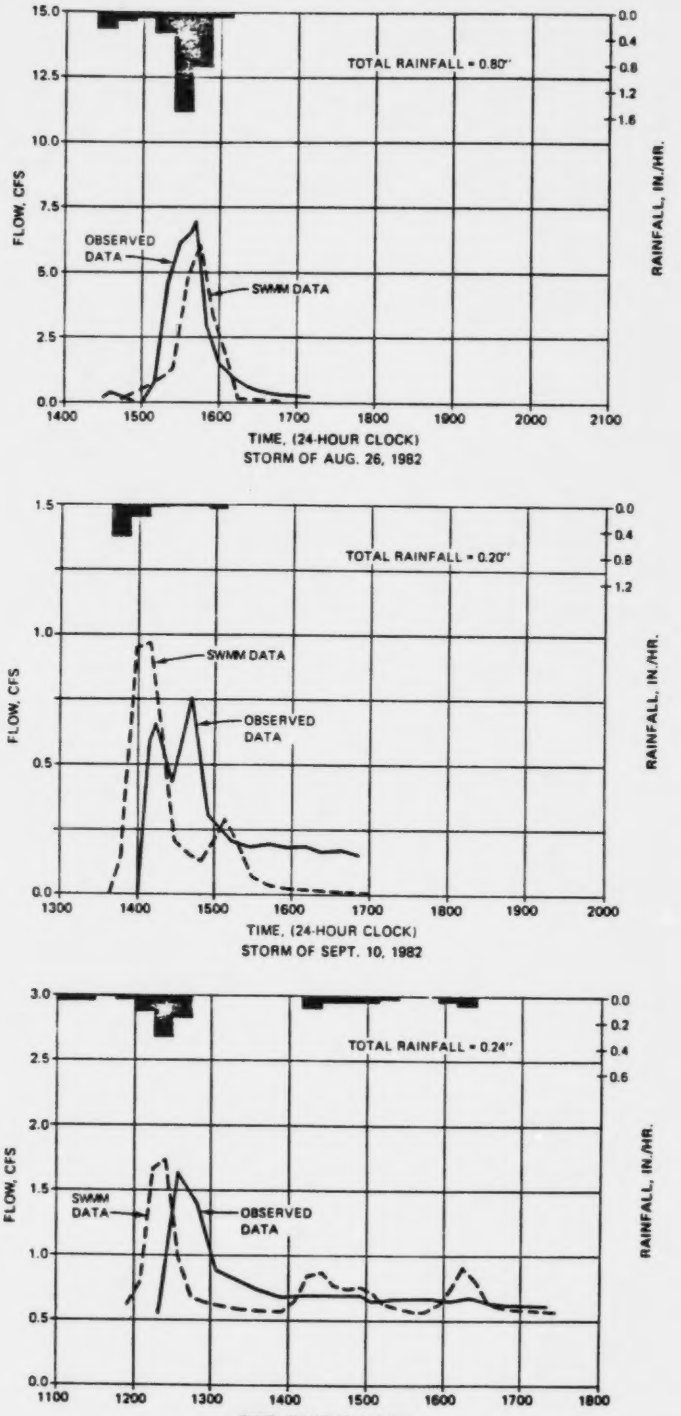


FIG. 2-4 J. L. YOUNG BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION

modeled basin is not correctly represented by the measured data. In most cases, rainfall measurements were conducted near the outfall point of the basin, rather than in the middle of the basin.

Despite the discrepancies mentioned above, the model results are good. No adjustments in model input parameters were made to obtain these simulations.

Wilder Ditch Basin - The largest of the five basins studied, this basin has a land area of 193.9 acres and contains single family residential, commercial and open space land uses. Approximately 39 percent of the land area is impervious and drainage occurs through a system of concrete pipes and ditches.

The discrete storm event data obtained on August 19, 1982, January 20, and January 21, 1983 was used for the calibration and verification of SWMM in the basin. The characteristics of these events are presented in Table 2-3.The observed and computed outflow hydrographs for each of these events are presented in Figure 2-5.

SWMM was initially calibrated using the data obtained on August 19, 1982. Although this storm is relatively small, 1.42 inches of rain was measured during a storm on the preceding day. The outflow of 2.4 cfs resulting from the earlier storm was still occurring when the storm began on the 19th of August. It was therefore modeled as a constant base flow. As shown, the shapes of the observed and computed hydrographs are similar. The

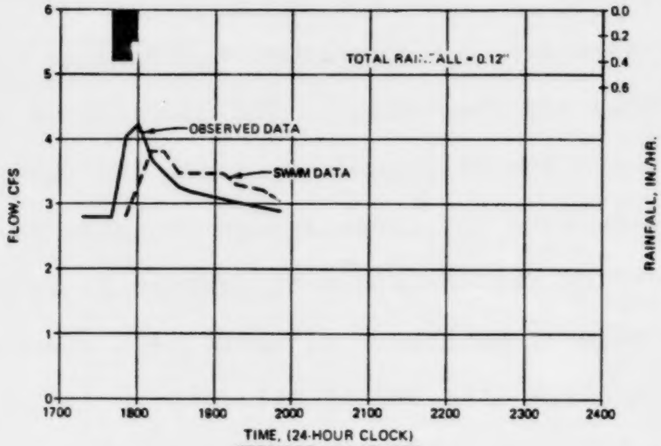
TABLE 2-3. WILDER DITCH - STORM CHARACTERISTICS FOR HYDRAULIC CALIBRATION

Storm date	Total rain, in.	Duration, hr.	Dry weather before storm, days	Intensity, in/hr		Base flow, cfs.
				Peak	Average	
August 19, 1982	0.12	0.3	0	0.40	0.36	2.38
January 20, 1982	0.70	5.5	38	0.84	0.13	0.65
January 21, 1982	0.45	3.0	0	0.72	0.15	1.21

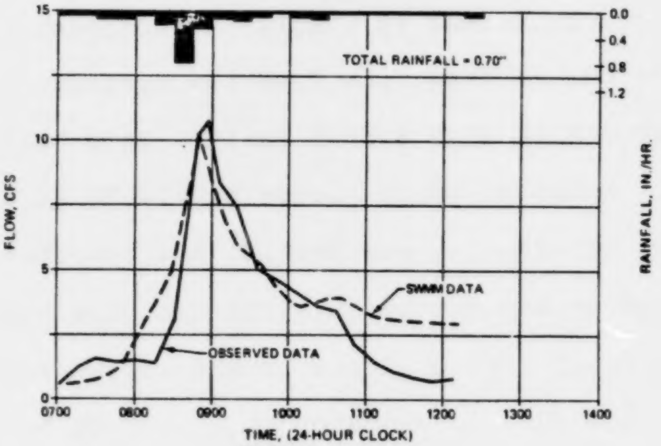
more rapid decrease in the observed outflow hydrograph probably is the result of the declining base flow from the previous storm. This base flow must be assumed as a constant for input to SWMM.

Initial model simulations for the two January storm events yielded hydrographs with peak flow rates and outflow volumes much greater than the observed data. It has been observed, however, that the ditch along the south side of West Wilder Avenue backs up and flows into a low, boggy area east of North Lincoln Avenue. In addition, some ditches in this basin become clogged with vegetation and debris to the extent that capacity is not sufficient to handle flows from larger storm events. In order to model this situation, in-system storage was added.

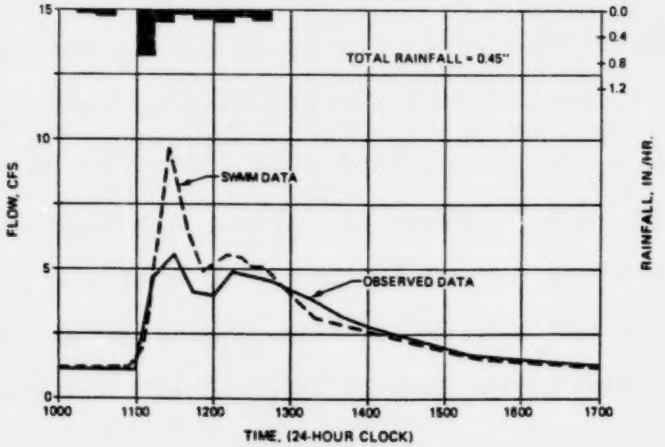
The capacity of storage was estimated using topographic maps. A flow of 2 cfs was set above which the ditch capacity



STORM OF AUG. 19, 1982



STORM OF JAN. 20, 1983



STORM OF JAN. 21, 1983

FIG. 2-5 WILDER DITCH BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION

would be exceeded and the storage volume would be filled. As shown in Figure 2-5, the simulated hydrographs for the two January storms are reasonable. The addition of storage has no effect on the storm of August 19, since the rainfall and resulting flow were not large enough to cause an overflow. The measured outflow for the storm of January 21 is somewhat lower than the simulated outflow. As mentioned, overflow from the ditch to the boggy area can be affected by debris in this system. In addition, the analysis of ditches as a stormwater runoff control described in Chapter 3 indicates infiltration from pervious ditches removes flow from the system.

North Jesuit High School Basin - This basin has a land area of 29.5 acres of which approximately 37 percent is impervious. Both low-density residential and institutional land uses are contained within the basin. The drainage system consists entirely of reinforced concrete sewers which range in size from a minimum of 18 inches in diameter to a 38 inch by 60 inch elliptical section. Precipitation information for this basin was obtained from a rain gage located nearby in Horizon Park.

The calibration and verification of SWMM was accomplished in the basin using storm event data collected on July 28, August 19, August 23, and December 12, 1982. Although discrete flow data were obtained during the latter event, the continuous runoff hydrograph was not available. The characteristics of each of these events are summarized in Table 2-4. The measured and

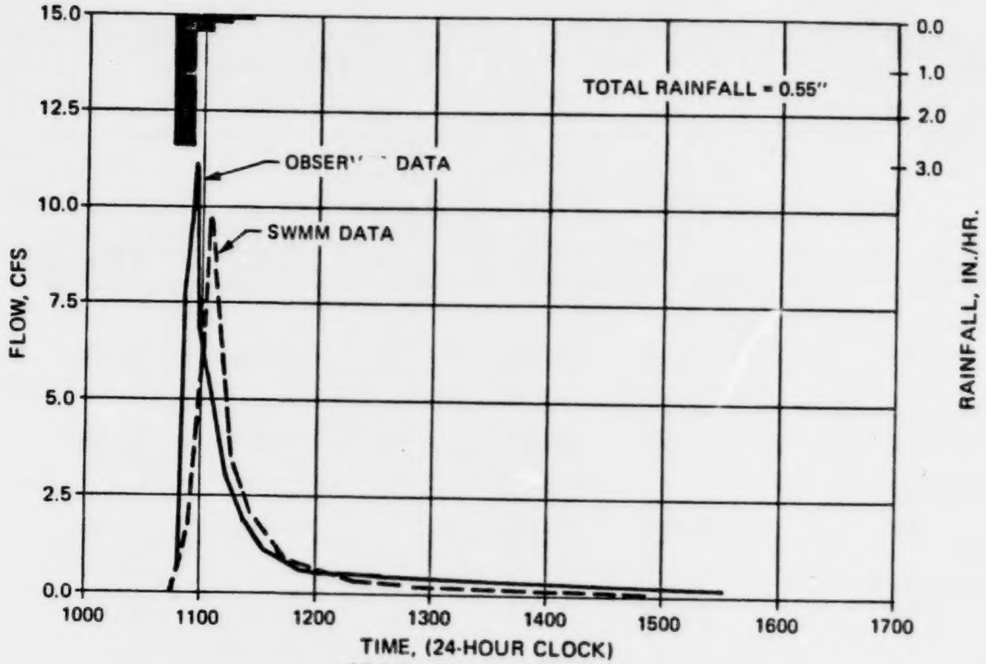
TABLE 2-4. NORTH JESUIT HIGH SCHOOL - STORM CHARACTERISTICS FOR HYDRAULIC CALIBRATION

Storm date	Total rain, in.	Duration, hr.	Dry weather before storm, days	Intensity, in/hr Peak	Intensity, in/hr Average	Base flow, cfs.
July 28, 1982	0.55	0.66	1	1.80	0.82	0.02
August 19, 1982	0.12	0.33	0	0.40	0.36	3.48
August 23, 1982	0.07	0.42	4	0.18	0.17	0.09
December 12, 1982	1.25	1.33	30	2.72	0.94	0.0

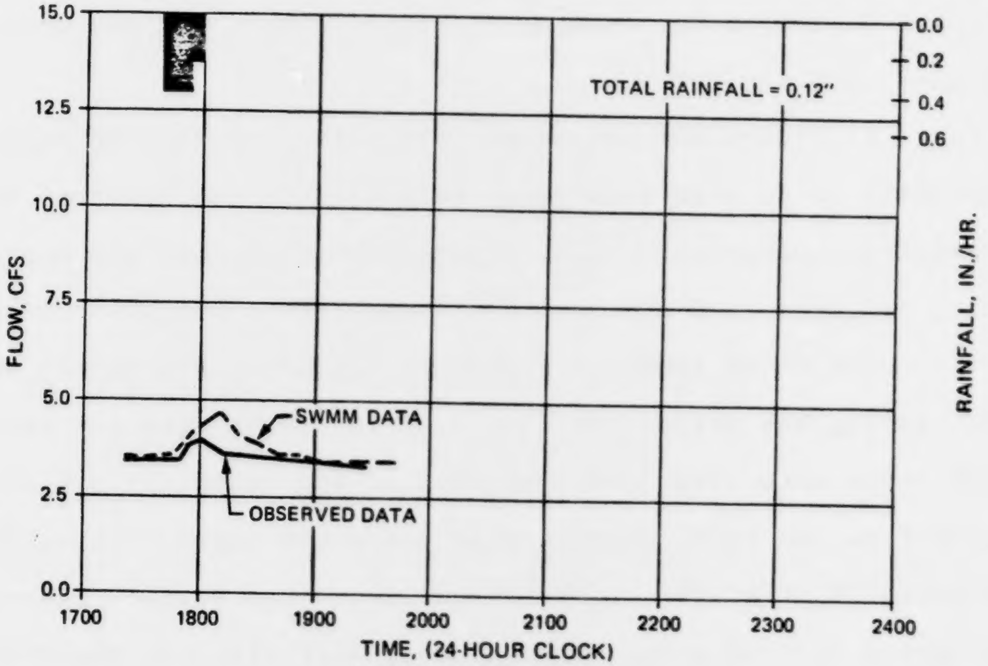
computed outflow hydrographs for these events are compared in Figures 2-6 and 2-7.

As Figure 2-6 indicates, the brief but very intense rainfall of July 28 is clearly reflected in the observed and computed hydrographs. The calibration shows that the outflow data computed by SWMM essentially duplicates the measured data.

The storm event of August 19 was brief and fairly small; but during the preceeding day, 1.42 inches of rain had fallen on the basin resulting in a base flow of approximately 3.5 cfs. This flow was still occurring as the event began on the 19th (see discussion of Wilder Ditch hydraulics). The outflow hydrograph computed by SWMM compares reasonably well with the observed

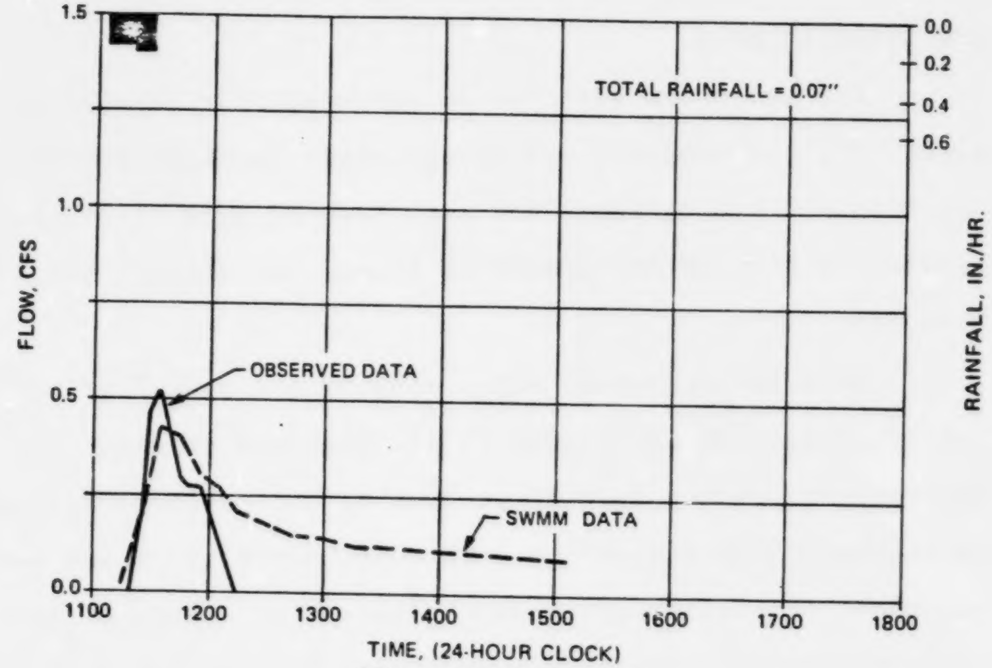


STORM OF JULY 28, 1982

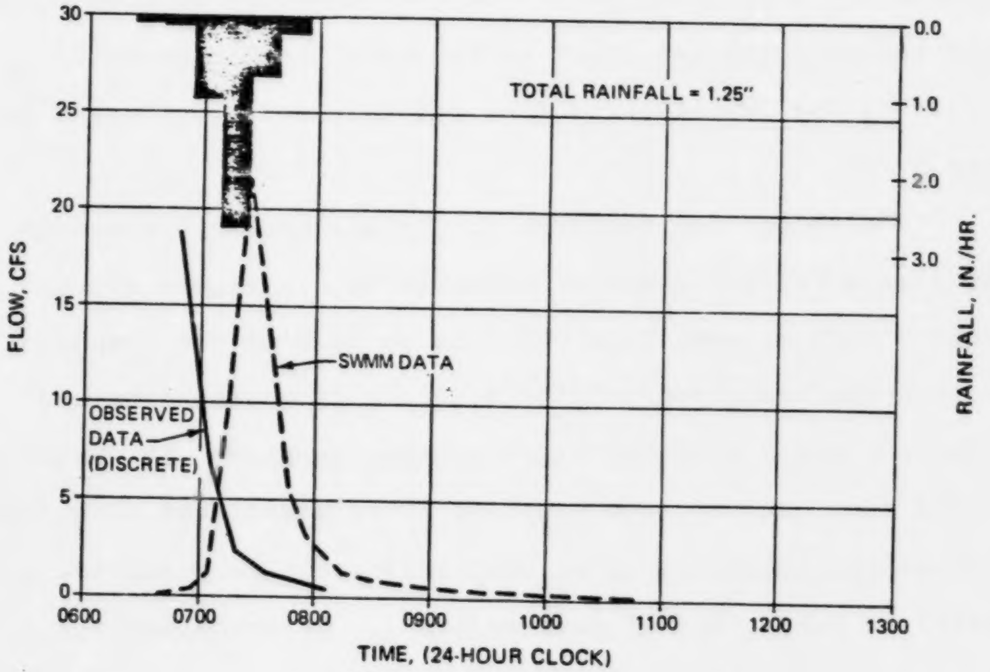


STORM OF AUG. 19, 1982

FIG. 2-6 NORTH JESUIT HIGH SCHOOL BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION



STORM OF AUG. 23, 1982



STORM OF DEC. 12, 1982

FIG. 2-7 NORTH JESUIT HIGH SCHOOL BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION (CON'T)

hydrograph, although the predicted peak is somewhat greater than the observed peak.

For the storm event which occurred four days later on August 23, the computed hydrograph again compares favorably with the measured hydrograph. The magnitude of this rainfall is similar to that of the preceding storm, but without the large base flow.

Both the magnitude and timing of the peak flow as predicted by SWMM correspond to the measured values. The reproduction of the initial portion of the measured hydrograph is particularly impressive for this small storm where the measured amount of rainfall approaches the detection limits of the gaging instrumentation. The slowly declining tail of the predicted hydrograph results in part from the base flow of 0.09 cubic feet per second which was input to the model as a constant. In reality, the initial base flow may have dropped to zero during the event.

Although the measured continuous hydrograph was not available for the event of December 12, in Figure 2-7 the salient points such as peak flow and time to peak of the computed hydrograph are compared with discrete flow data obtained simultaneously with the water quality samples. It should be noted that the recorded sampling times associated with these values are questionable as they indicate a peak outflow occurring prior to the period of peak rainfall. Nevertheless the shape of the SWMM hydrograph and the peak value are similar to the hydrograph defined by the discrete values.

Charter and Harding Basin - The basin contains 42.2 acres of land of which 31 percent is impervious. Land use within the basin is largely residential and the entire basin is served by a storm sewer system consisting of concrete pipes which range in diameter from 18 to 36 inches. Runoff is collected and transfered to the sewers by means of either swales, curb gutters or grass gutters. Rainfall quantities used during the hydraulic analysis of this basin were obtained from a gage located about 35 feet from the outflow control structure.

Discrete sampling was conducted in this basin during storm events on September 8, 1982; January 20, and February 2, 1983. Due to equipment malfunctions, continuous outflow hydrographs were not recorded during the latter two events. Therefore hydraulic calibration and verification was accomplished using data obtained during the discrete storm of September 8 and during a storm on June 25, 1982. The characteristics of these two events are summarized in Table 2-5.

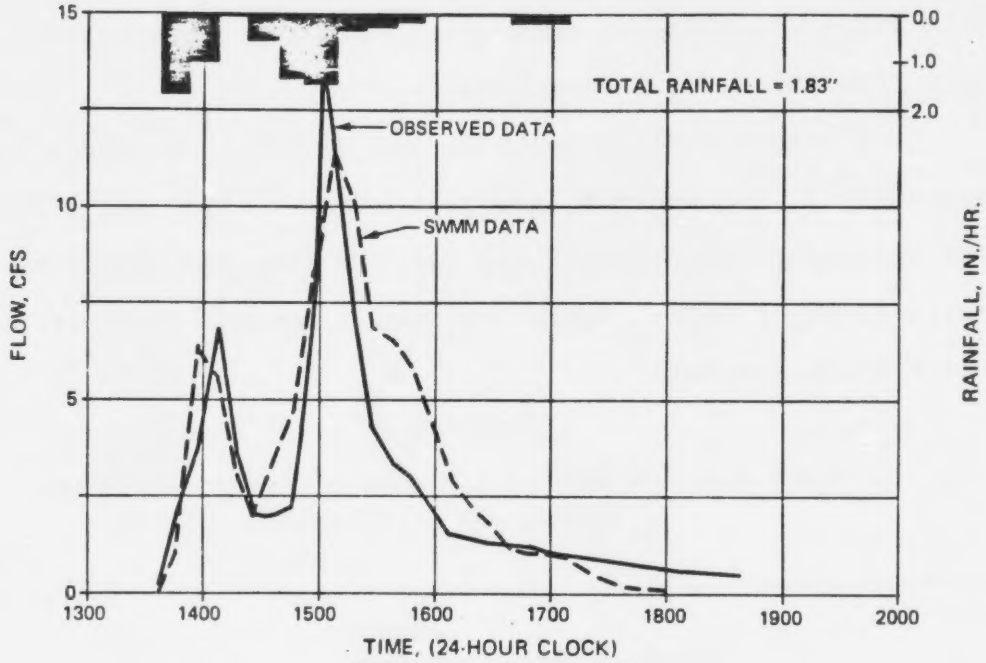
As shown in Figure 2-8, for the first of the events, the magnitude of peak outflows as predicted by SWMM correspond closely to the measured outflows. For the storm of September 8, the initial calibration resulted in predicted peak and average flows significantly less than the observed values. The rainfall record indicated that 8.39 inches of rain had fallen during

TABLE 2-5. CHARTER & HARDING - STORM CHARACTERISTICS FOR HYDRAULIC CALIBRATION

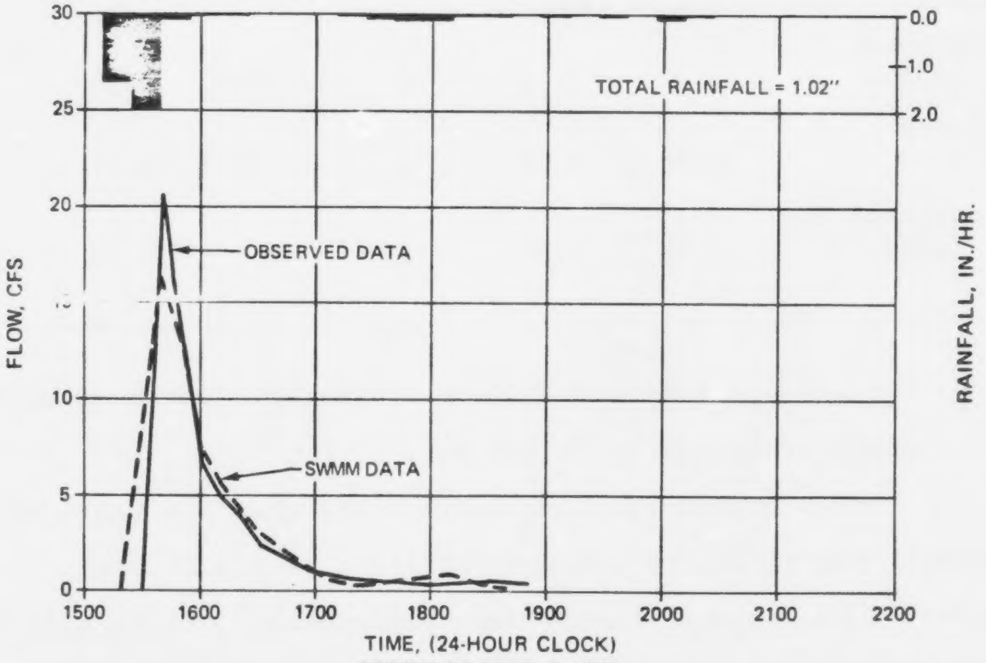
Storm date	Total rain, in.	Duration, hr.	Dry weather before storm, days	Intensity, in/hr Peak	Intensity, in/hr Average	Base flow, cfs.
June 25, 1982	1.83	3.5	1	1.64	0.52	0.25
September 8, 1982	1.02	4.8	0	1.92	0.21	0

the previous 3 days with 2.74 inches recorded on the preceding day. The unusually wet conditions may have resulted in saturated soil conditions and reduced infiltration capacity in pervious areas. To reflect this probable saturated soil condition, another SWMM run was made with the maximum infiltration rate (which is typically set at 3.0 inches per hour) reduced to the typical minimum value of 0.5 inches per hour. The outflow hydrograph produced by this run is shown in Figure 2-8. Although the magnitude of the computed peak flow is slightly less than the observed peak flow, the predicted hydrograph compares well with the observed outflow hydrograph.

Norma Park Basin - This highly developed basin consists primarily of commercial land uses. Approximately 90 percent of the 46.6 acre land area is impervious with stormwater runoff directed through elliptical concrete storm sewers to the ditches which run parallel with Dale Mabry Highway. Stormwater has been reported to accumulate in some ditch sections because ditch invert elevations are below the culvert inverts. The rainfall



STORM OF JUNE 25, 1982



STORM OF SEPT. 8, 1982

FIG. 2-8 CHARTER & HARDING BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION

data for this basin was obtained from a gage located about 12 feet from the outfall structure.

Discrete quality sampling was conducted in Norma Park on September 7, September 8, and September 21, 1982 and therefore the corresponding rainfall and outflow data were utilized for the calibration of SWMM. Table 2-6 summarizes the characteristics of these storm events.

TABLE 2-6. NORMA PARK STORM CHARACTERISTICS FOR HYDRAULIC CALIBRATION

Storm date	Total rain, in.	Duration, hr.	Dry weather before storm, days	Intensity, in/hr		Base flow, cfs.
				Peak	Average	
September 7, 1982	1.32	3.75	0	1.60	0.33	0.14
September 8, 1982	0.28	1.62	0	0.28	0.17	0.47
September 21, 1982	1.25	6.38	13	1.60	0.20	0.15

The observed and computed outflow hydrographs for each of these events are presented in Figure 2-9.

The measured hydrographs indicate that the basin does not exhibit the immediate response to storm events as might be expected from such a highly impervious basin. It appears that the ditches are responsible for a loss of total runoff volume as well as attenuation of flow. In-system storage probably occurs

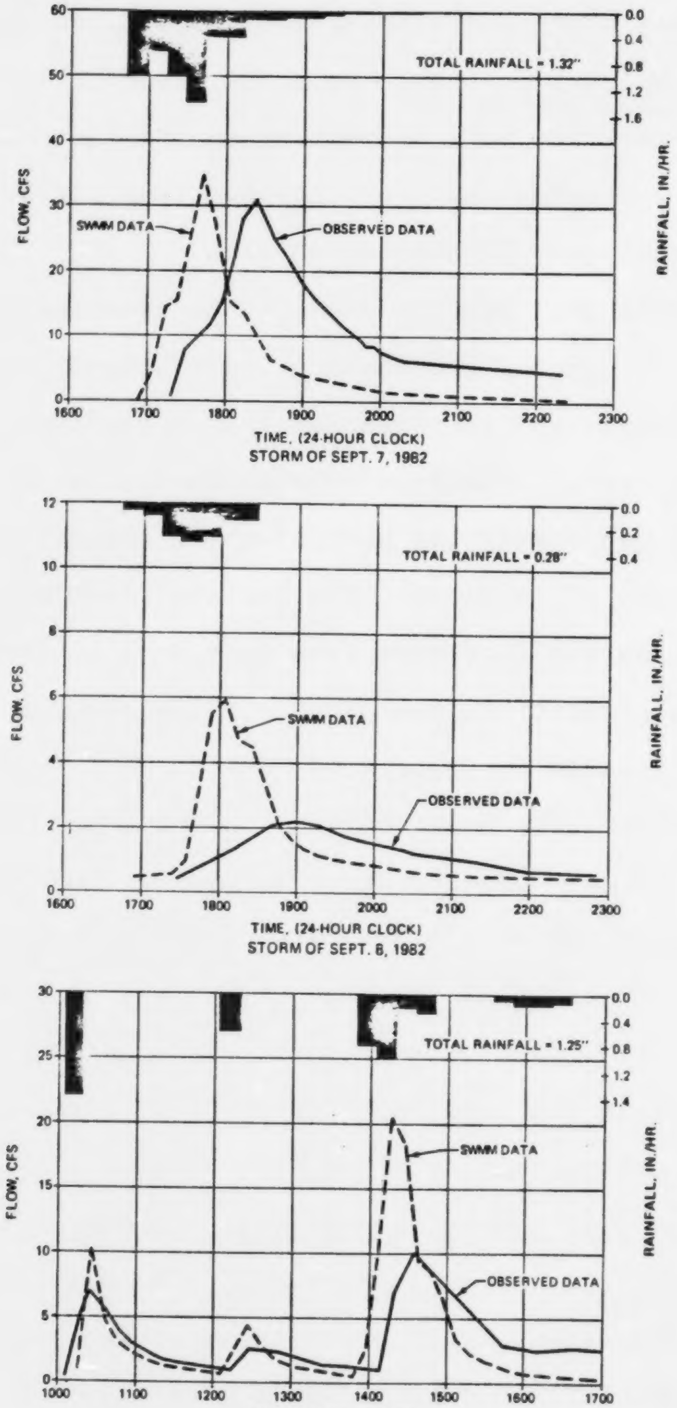


FIG. 2-9 NORMA PARK BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION

in many areas where the ditches slope uphill or where the downstream culvert is above the ditch invert. In addition, the analysis of ditches as a stormwater runoff control, described in Chapter 3, indicates that infiltration from pervious ditches removes significant quantities of flow.

Because SWMM was not designed to describe the hydraulically complex processes of open channel flow through undercut ditches and infiltration from a pervious transport system, the initial calibration was poor. The calibration was improved by decreasing the modeled basin impervious area from 90 to 60 percent. In addition, the surface retention storage of pervious areas was increased from 0.18 to 3.00 inches to simulate the storage of water in the ditches. The results of these model runs are presented in Figure 2-9.

Despite these modifications to the model input, the comparison of computed and observed hydrographs in the basin is not as good as was obtained in the other four basins. Although the predicted peak flows are too high, SWMM does provide a general description of the hydraulic response to the basin. It also highlights the important impact of undercut ditch systems on volume and attenuation of stormwater runoff.

Summary of Hydraulic Modeling. Through the SWMM calibration and verification process, the following factors, listed in decreasing order of importance, were determined to have a significant influence upon runoff generation:

1. Rainfall
2. Infiltration
3. Basin and transport characteristics
4. Ditch systems
5. In-system storage

It should be stressed that the calibration and verification runs were performed with a minimum of changes in input parameters. Only some of the parameters listed above were modified, and these modifications were only required in special instances. A description of each parameter and its impact on stormwater flows follows.

Rainfall - This parameter was not adjusted during calibration, as it was measured and used as direct model input. It is obvious that rainfall, being the driving force of runoff, is a major factor in runoff generation. The development of rainfall inputs under design conditions was presented elsewhere (6).

Infiltration - For all storm events except one, the Horton equation with initial and final rates of 3.0 and 0.5 inches per hour respectively, resulted in satisfactory model simulations. For the one case mentioned, during an extremely wet period in June, the initial rate was reduced to 0.5. In most situations, however, the initial rate of 3.0 appears reasonable.

Basin and Transport System Characteristics - These values were fixed as model inputs and were not changed during the calibration process except for Norma Park. The percent imperviousness was decreased in Norma Park to simulate the effect of infiltration from undercut ditches. Of all the basin and transport system characteristics, the percent of impervious area is the major factor affecting the magnitude of the peak flow and volume of runoff.

Ditch Systems - As shown for the Wilder Ditch and Norma Park basins, ditches can allow storage of water behind undercuts or debris and also infiltration through a pervious bottom. Ditch systems are an important urban runoff control device which will be evaluated extensively in Chapter 3.

In-System Storage - Surface storage by ponding in low-lying areas such as in the Wilder Ditch Basin was shown to be effective in reducing outflow runoff volume. The effect of retention ponds on stormwater flows will be evaluated in Chapter 3.

SWMM Calibration - Water Quality. After SWMM was hydraulically calibrated to describe the measured outflows in each basin, the computed outflow concentrations were compared with the quality data obtained during the three discretely sampled events. It is difficult to model stormwater quality due to its variability. This variability can be a function of many parameters such as time of year, day or week; origin of the storm system; characteristics of the storm; days of dry weather since the last storm; and transient land uses such as construction and road work. SWMM represents only a subset of the possible influences on stormwater quality. This subset consists of readily identifiable and easily defined parameters. The objective is to calibrate the model so that it represents approximately mean stormwater quality. Given this approach, observed measurements are expected to vary somewhat about the predicted condition.

In this application of SWMM, stormwater quality is computed based solely upon the parameters specified as input to the Runoff Block. The amount of solids available for wash off during a given event is computed using a constant accumulation rate of dust and dirt per length of gutter which is specific to each land use and the number of days since the last removal of dust and dirt by street cleaning or by a previous storm event. The predicted outflow BOD consists of soluble and non-soluble portions. The soluble portion is computed for each subcatchment using a constant which is characteristic of that land use. The non-soluble BOD as well as coliform are computed as a constant percent of the accumulated dust and dirt regardless of land use. Therefore as modeled by SWMM, the concentrations of the three pollutants are dependent upon the amount of available dust and dirt.

In calibrating the quality portion of SWMM, the dust and dirt accumulation rates in each basin were set to obtain the best comparison of computed and measured outflow suspended solids concentrations. The BOD and coliform multipliers were adjusted subsequently. During the SWMM water quality calibration process, the philosophy employed was to determine the overall best fit parameters for each basin. For the most part, parameters were not changed from storm event to storm event within a specific basin.

The SWMM input parameters used to produce the final calibrations in each basin are given in Table 2-7 by land use. Also presented are SWMM default values, developed primarily from data collected in Chicago, San Francisco, and other heavily urbanized areas. A comparison of the best fit values in Tampa with the SWMM default values suggests the following:

1. Dust and dirt accumulation rates are generally much lower in Tampa than the model default values.
2. The fraction of soluble BOD per pound of solids accumulation is higher in Tampa.
3. The amount of coliform bacteria is highly variable but appears to be the same or slightly higher in Tampa.

Based on this information it appears that the more suburban nature of Tampa (i.e. less industrialized, more grass land and vegetation), may result in less solids accumulation but higher availability of organic matter per unit of solids. Additional comparisons of stormwater quality in Tampa with that of other areas will be presented later in this Chapter.

The stormwater quality simulations indicated that the fraction of suspended solids that is non-soluble BOD was from 5 to 60 percent. However, BOD content of street sweeper sediments was found to be only 0.07 percent of the solids during the NURP

TABLE 2-7. STORMWATER QUALITY PARAMETERS USED IN CALIBRATION OF SWMM

Basin	Land use(1)	Percent land use in basin	Dust and dirt per 100 ft. gutter, lb	Soluble BOD per lb dust and dirt mg/gm	Non-soluble BOD(2)	Coliform multiplier(3)
J.L. Young	2	100	0.05	20	0.2	5.0
Wilder Ditch	1	51	0.04	50	0.6	1.0
	2	24	0.18	77		
	3	7	0.26	30		
	4	17	0.08	50		
North Jesuit	1	58	0.5	25	0.25	1.0
	4	42	2.2	38		
Charter & Harding	1	90	0.04	10	0.10	5.0
	4	10	0.08	10		
Norma Park	1	7	0.01	5.0	0.05	5.0
	3	93	0.02	7.7		
SWMM Default Values	1	-	0.7	5.0	0.05	1.0
	2	-	2.3	3.6		
	3	-	3.3	7.7		
	4	-	1.5	5.0		

1. Single family residential = 1, multi family residential = 2, commercial/institutional = 3, open space = 4.
2. Fraction of dust and dirt that is non-soluble BOD.
3. Multiplier used for coliform calibration.

sediment analysis of May 16, 1983. While this value is obtained from only two samples, a two orders of magnitude difference suggests that significant difference would be found for larger sample sizes as well. This is especially true since the two samples gave similar values (0.04 and 0.10 percent). By comparing the BOD content of the street sweeper sediments and the stormwater, it appears that BOD in stormwater is largely soluble and not associated with the solids. The discrete data which showed that peak BOD concentrations frequently do not coincide with peak suspended solids concentrations supports this further. In addition, particle size distributions showed that street sweeper sediments are largely comprised of sandy material which is typically inorganic.

The following discussion focuses on the SWMM quality calibration for each basin. The water quality characteristics observed during the discrete storm events are summarized in Table 2-8.

J.L. Young Water Quality - The measured and computed pollutographs for the three discretely sampled storm events are presented in Figures 2-10 through 2-12. During these three events, effluent concentrations of suspended solids and BOD were typically less than 50 mg/l and 20 mg/l, respectively. The measured coliform concentrations ranged from 5,000 to 170,000 MPN/100 ml. With the exception of coliform, for all three events the data computed by SWMM compares very well with the measured

TABLE 2-8. RUNOFF QUALITY CHARACTERISTICS - DISCRETE STORM EVENTS (1)

Basin	Storm date	Suspended solids, mg/l		BOD$_5$, mg/l		Total coliform, MPN x 10^3/100 ml
		mean	range	mean	range	range
J.L. Young	Aug. 26, 1982	8	5-103	13	9-17	5-49
	Sept. 10, 1982	17	5-46	11	4-22	17-79
	Sept. 21, 1982	26	7-144	6	1-9	8-170
Wilder Ditch	Aug. 19, 1982	8	5-13	10	7-13	TNTC(2)
	Jan. 20, 1983	21	2-76	13	2-42	2-130
	Jan 21, 1983	44	6-89	70	4-9	23-170
North Jesuit	Aug. 19, 1982	31	9-71	20	10-25	TNTC(2)
	Aug. 23, 1982	20	9-59	24	2-49	4-8
	Dec. 12, 1982	212	43-338	23	4-29	8-540
Charter & Harding	Sept. 8, 1982	29	5-193	5	2-15	130-1,600
	Jan. 20, 1983	22	2-54	14	9-23	33-540
	Feb. 2, 1983	26	2-146	10	2-13	33-1,600
Norma Park	Sept. 7, 1982	11	5-108	10	5-20	33-350
	Sept. 8, 1982	6	5-12		(3)	8-130
	Sept. 21, 1982	41	5-95	25	12-34	2-540

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1. Data obtained from Runoff Characterization Water Quality and Flow Data, Task II.5 Report (2).
2. Too numerous to count.
3. Not available.

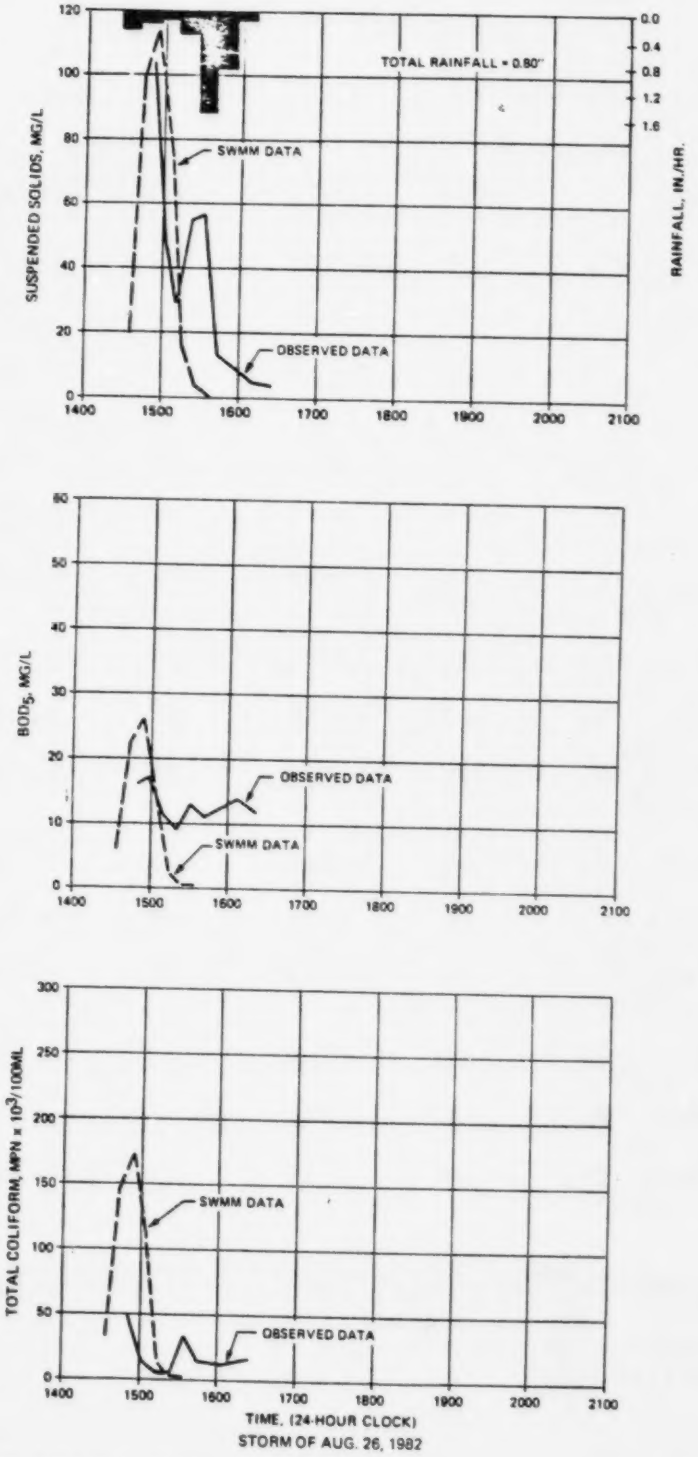


FIG. 2-10 J. L. YOUNG BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, AUGUST 26, 1982

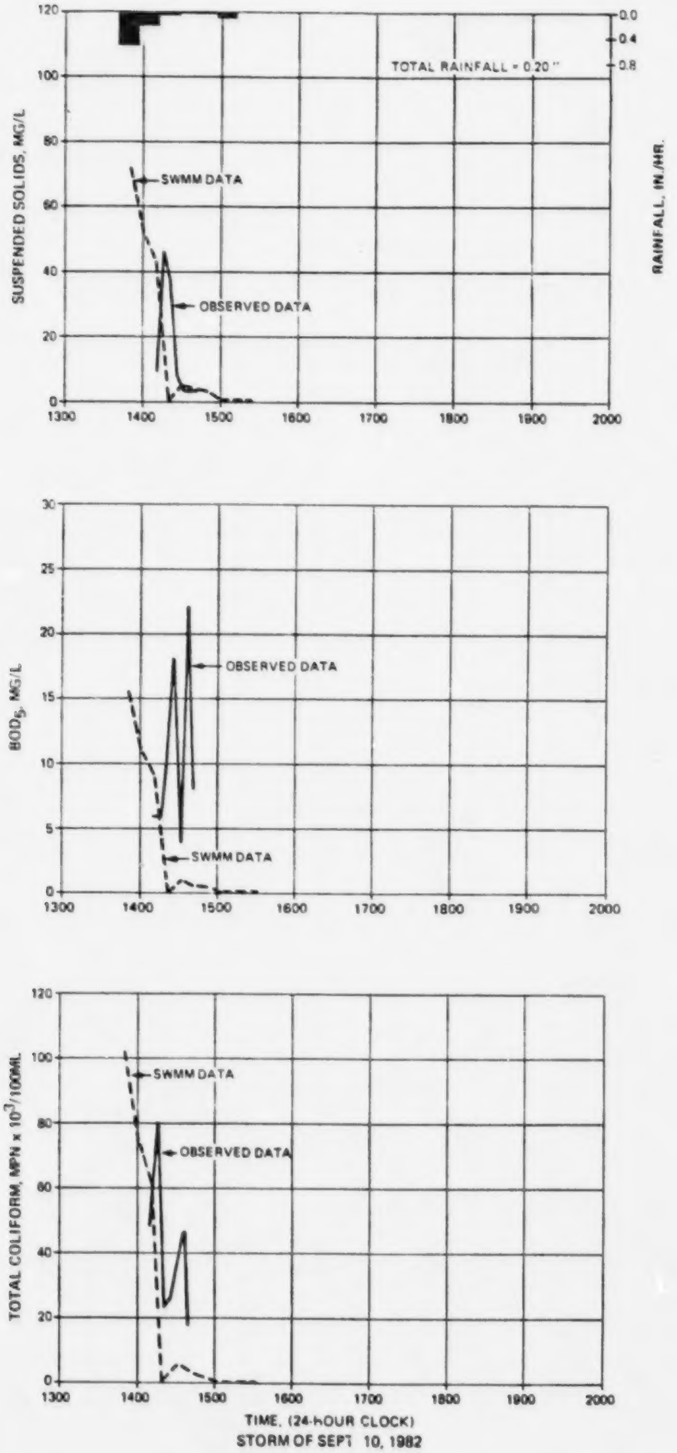


FIG. 2-11 J. L. YOUNG BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, SEPTEMBER 10, 1982

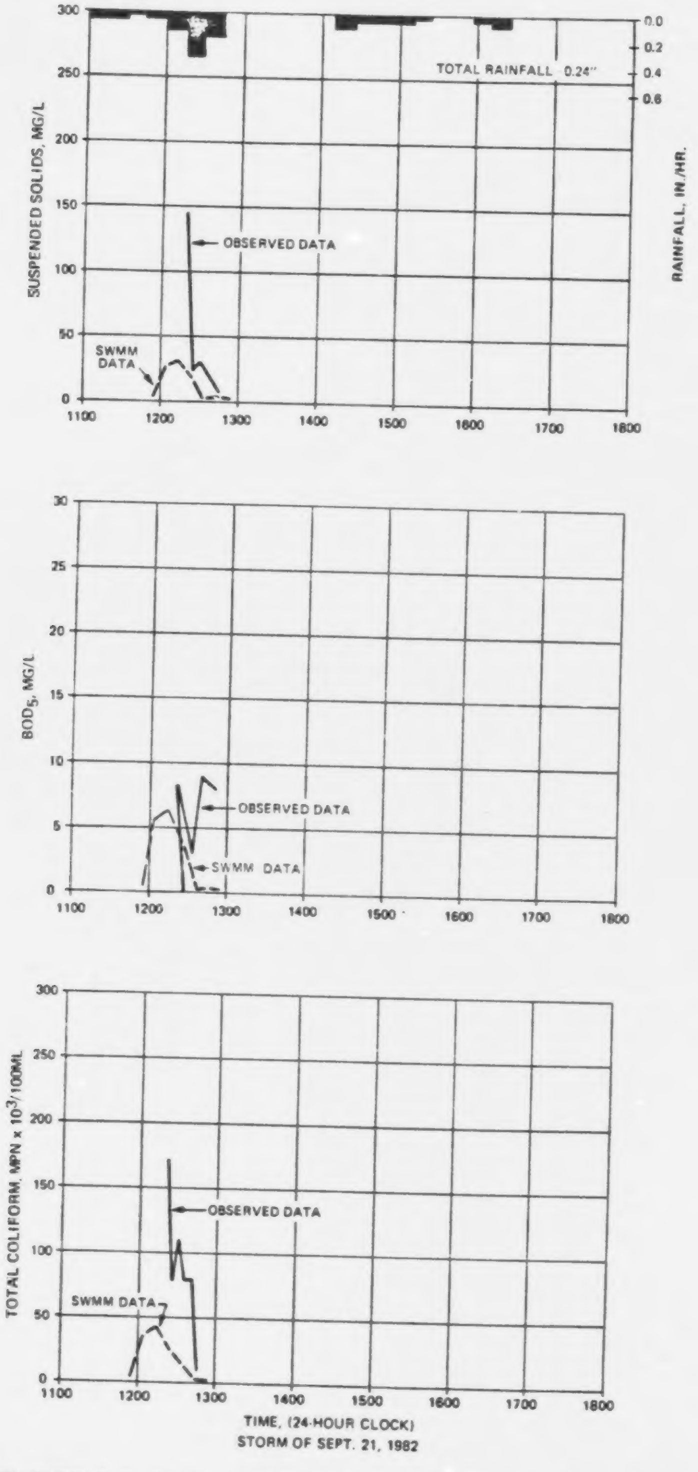


FIG. 2-12 J. L. YOUNG BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, SEPTEMBER 21, 1982

data. Of the three coliform simulations performed, one simulation predicts concentrations that are higher than the measured values, another predicts below the measured values and the third is relatively consistent with them. Therefore, in order to calibrate the SWMM, it would be necessary to adjust the coliform multiplier for each storm. Because of the large variability in coliform data, such a procedure was not pursued.

Wilder Ditch Water Quality - Figures 2-13 through 2-15 present the computed and measured pollutographs for the discretely sampled events in this basin. The measured effluent concentrations of suspended solids and BOD are typically less than 100 mg/l and 50 mg/l respectively, and the computed concentrations fall within the same range. Although coliform data were not available for the storm event of August 19, 1982, the storms in January exhibit widely ranging concentrations, which are reproduced well by SWWM for one of these events. In general, SWMM provides an adequate description of the variation in outflow concentrations in this basin.

The event on January 20 was preceded by 38 days without rainfall. Using a 38 day antecedent condition, the initial quality calibration yielded computed concentrations much greater that the measured concentrations. It was apparent that the computation of dust and dirt build-up as a linear function of time over this long period was not an accurate representation of the actual accumulation. Studies have indicated (8,9,10) that

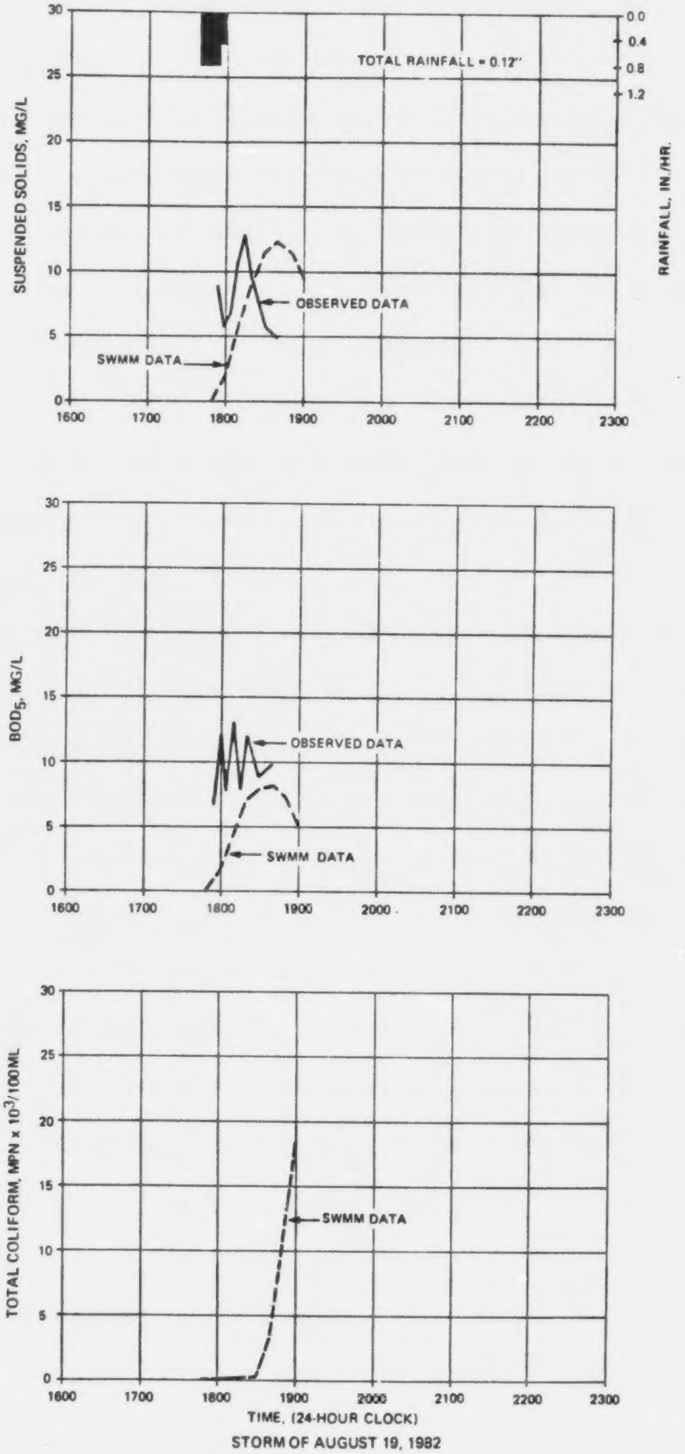


FIG. 2-13 WILDER DITCH BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, AUGUST 19, 1982

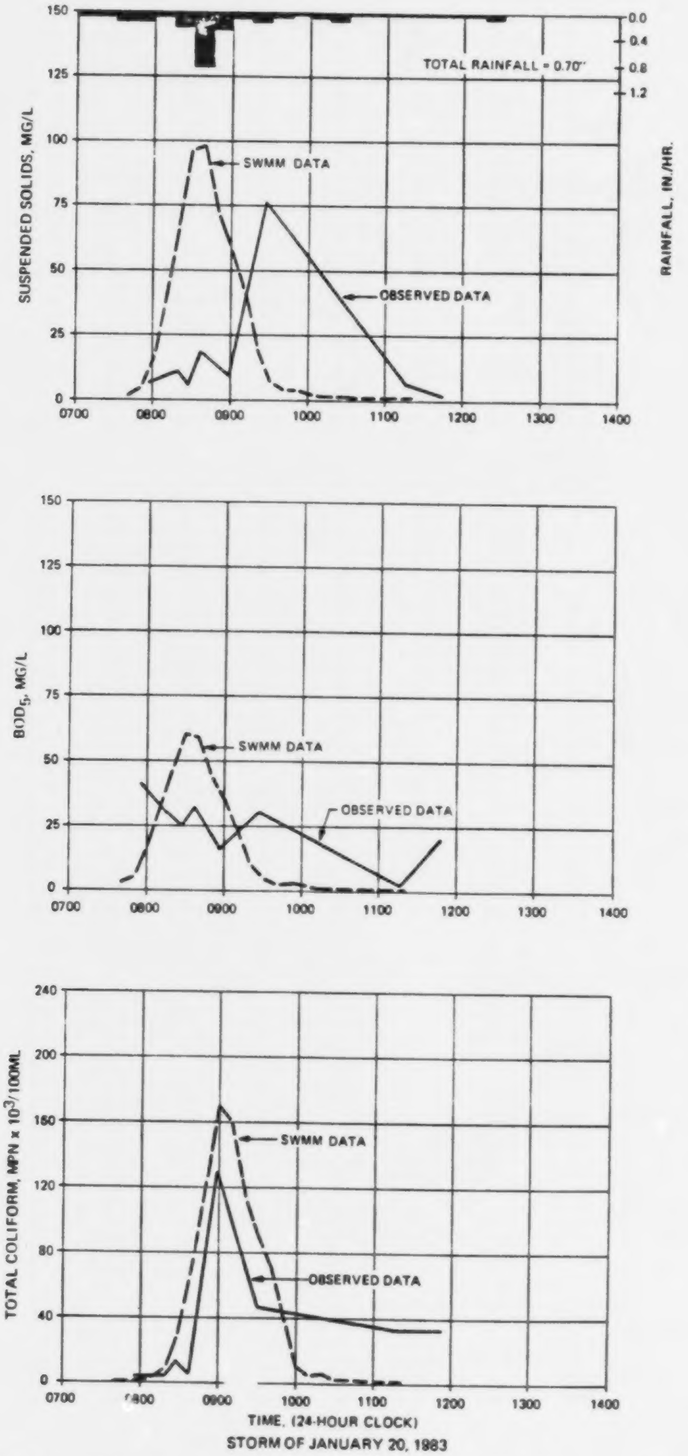


FIG. 2-14 WILDER DITCH BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, JANUARY 20, 1983

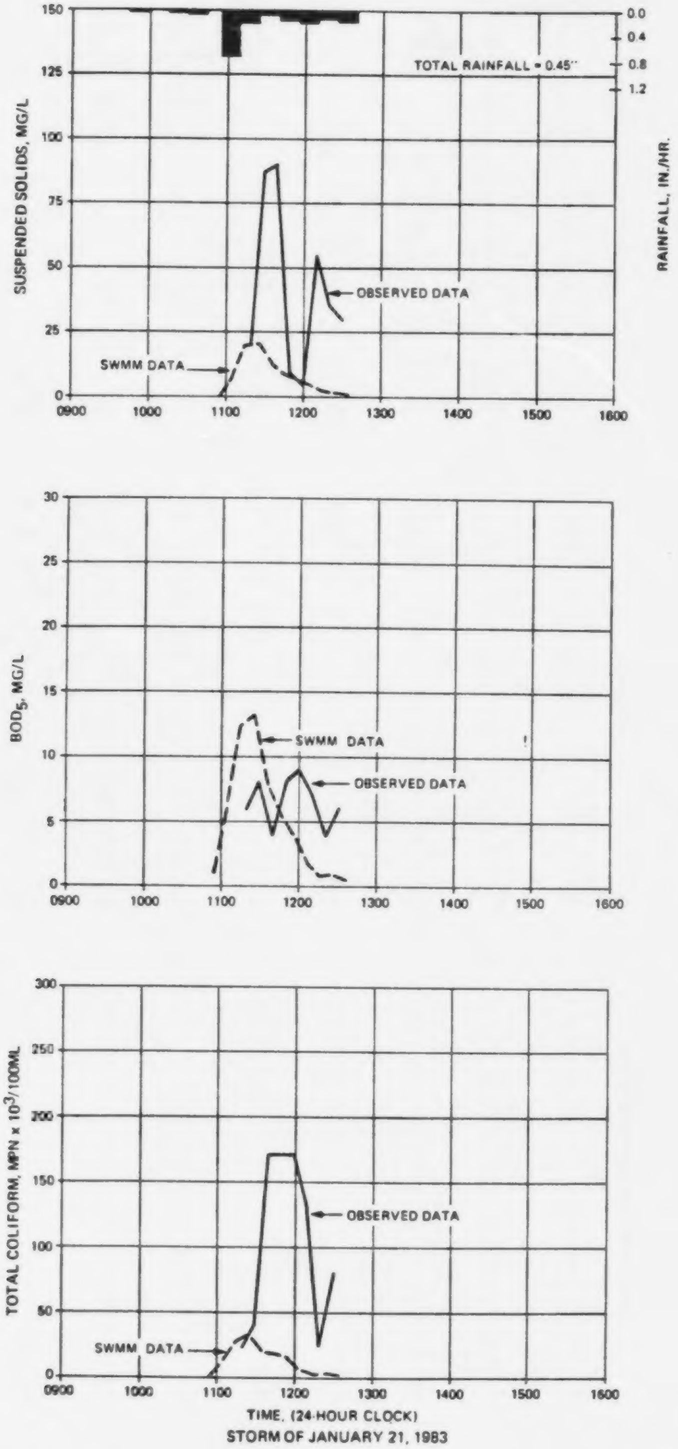
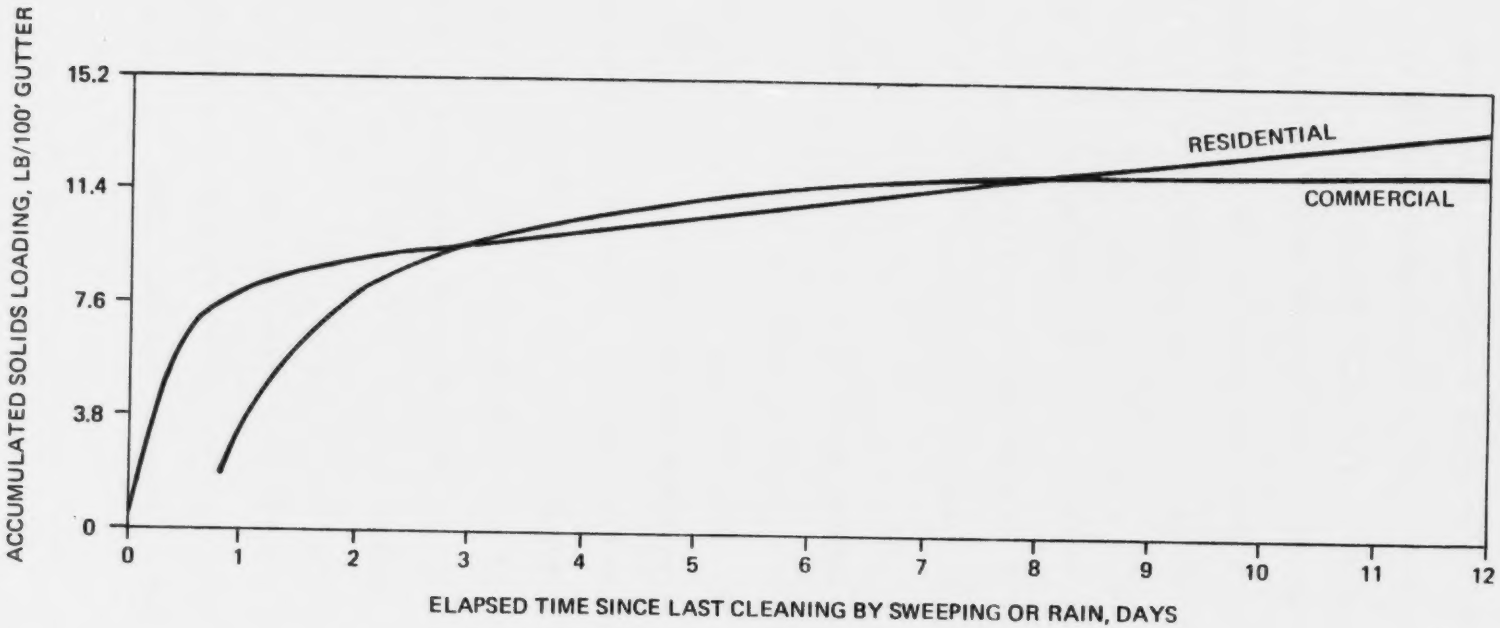


FIG. 2-15 WILDER DITCH BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, JANUARY 21, 1983

accumulation of solids often reaches a plateau after which additional build-up does not occur. To improve the initial calibration and develop a more realistic approximation of solids accumulation, SWMM was rerun for this event using an antecedent condition of six days. Six days was selected based on buildup curves from Sartor and Boyd (8) shown in Figure 2-16. These curves are for residential and commercial land uses of which Wilder Ditch and the rest of the Lower Hillsborough River Basin is primarily composed. The resulting improvement in quality calibration supports the assumption of non-linear solids accumulation.

North Jesuit High School Water Quality - The comparisons of the measured and computed quality data are presented in Figures 2-17 through 2-19. For the two events in August, the measured suspended solids and BOD concentrations are in the same range as the values observed in the J.L. Young and Wilder Ditch basins. The computed values also fall in this range, corresponding well with the measured data. The storm event of December 12 was preceded by 30 days without rainfall, so again an antecedent condition of 6 days was input to the model to better describe pollutant accumulation.

As shown in Figure 2-19, the suspended solids and coliform concentrations measured during this event were much greater than those observed during other discrete events. The



SOURCE: SARTOR AND BOYD, 1972, p. 206

FIG. 2-16 NON- LINEAR BUILDUP OF STREET SOLIDS

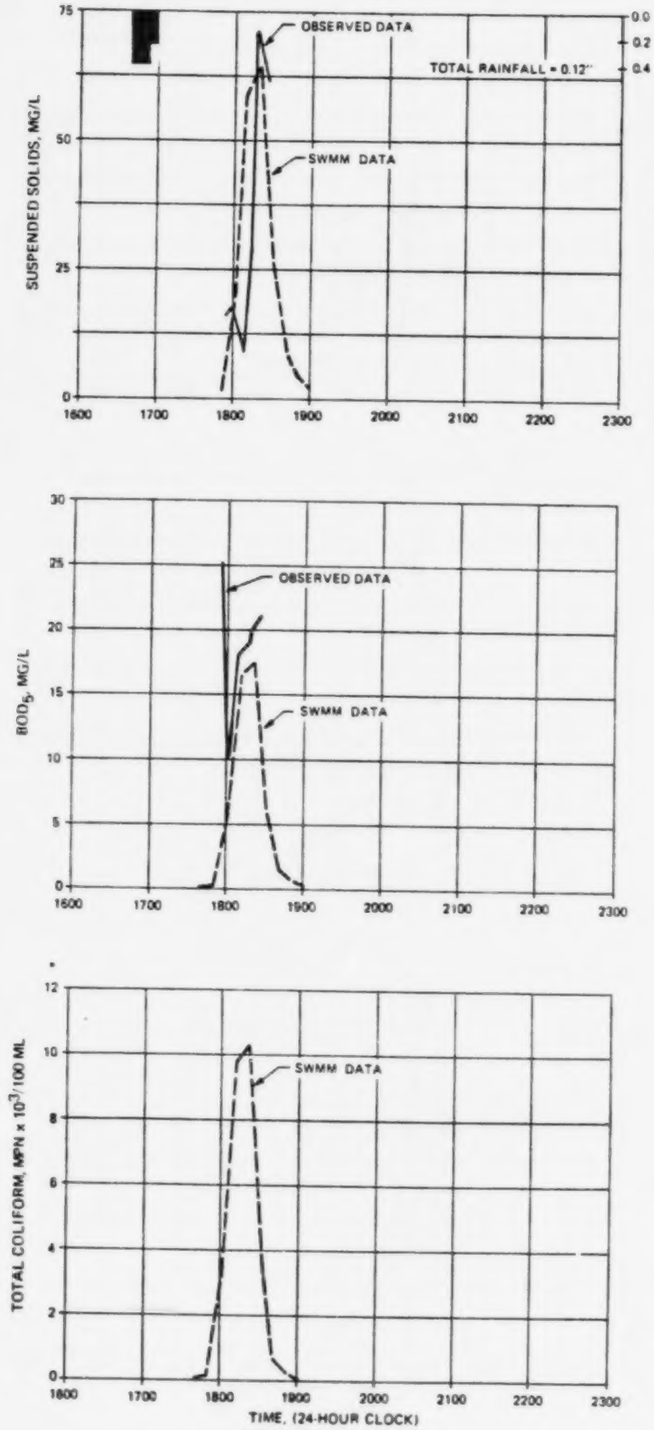


FIG. 2-17 NORTH JESUIT HIGH SCHOOL BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, AUGUST 19, 1982

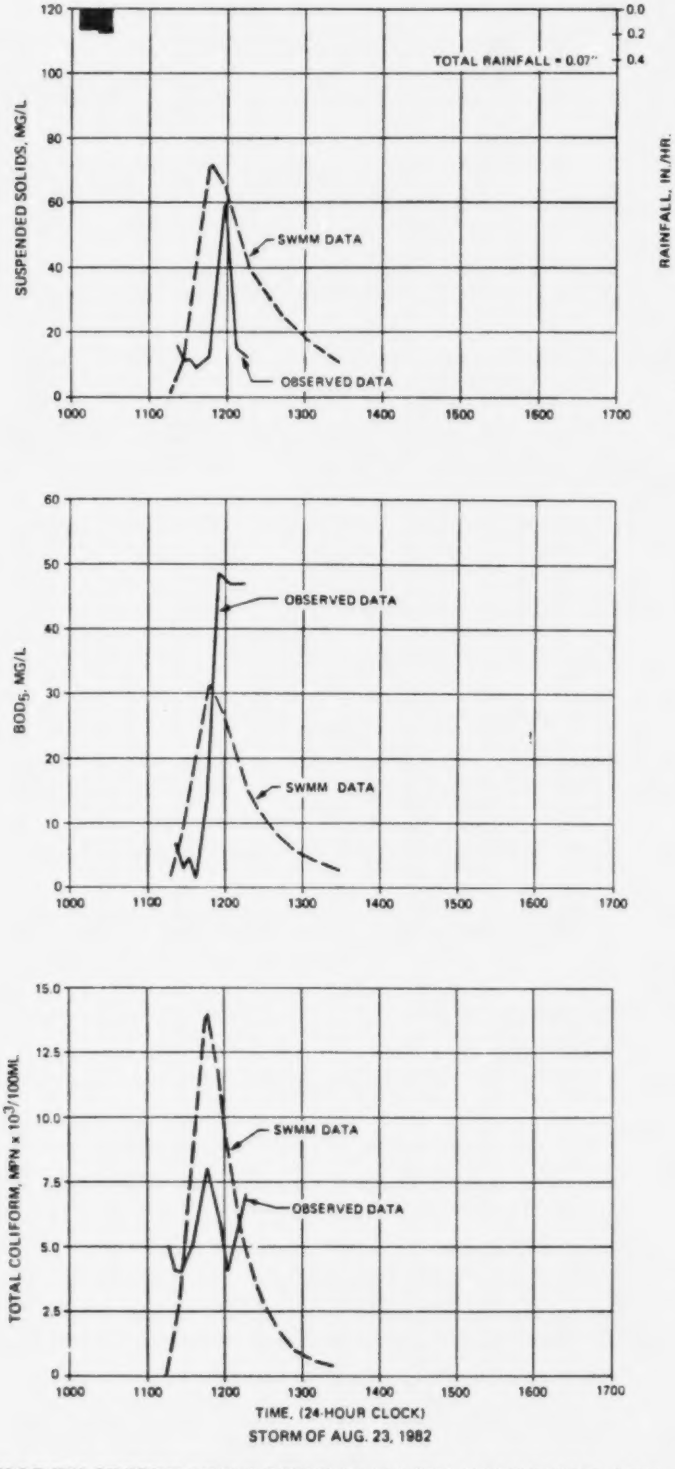


FIG. 2-18 NORTH JESUIT HIGH SCHOOL BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, AUGUST 23, 1982

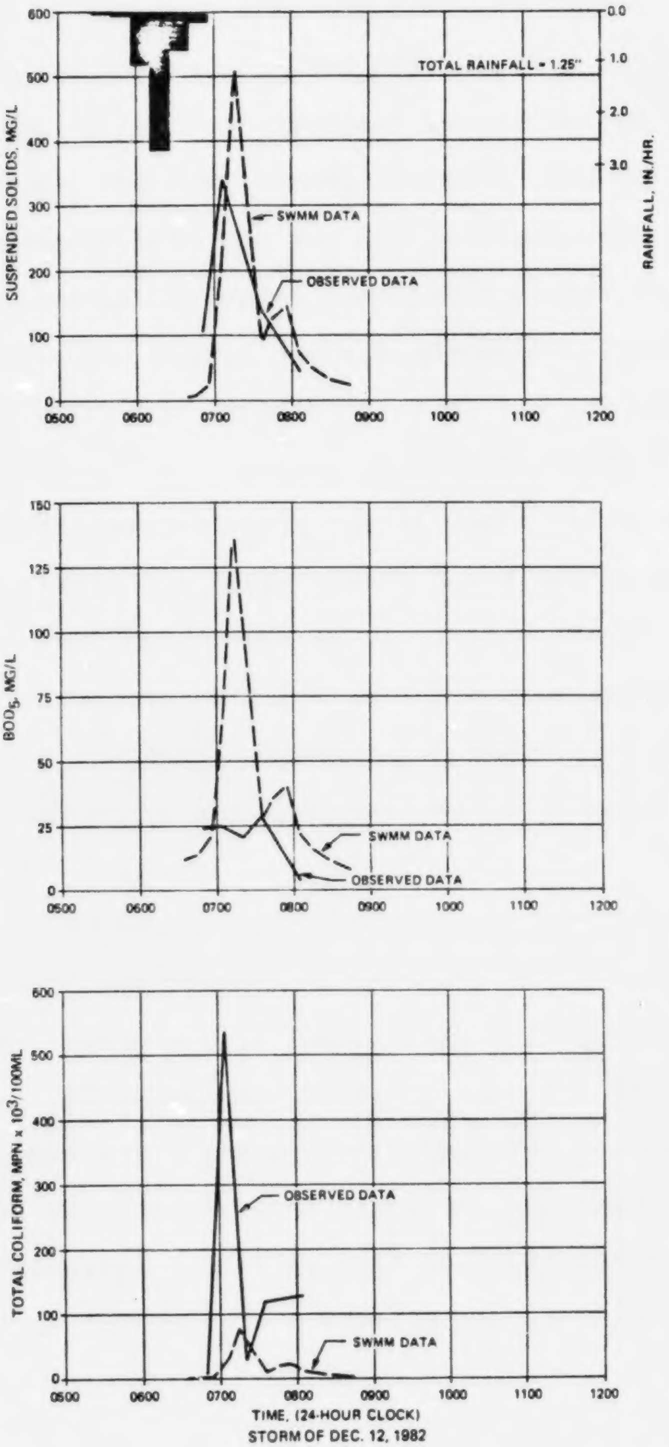


FIG. 2-19 NORTH JESUIT HIGH SCHOOL BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, DECEMBER 12, 1982

period of intense rainfall may have resulted in a large washoff of inorganic material. The SWMM pollutograph supports this hypothesis by indicating that peak pollutant concentrations coincide with peak rainfall intensity. The computed solids concentrations generally fit the observed concentrations. However, the strict functional relationship between solids and BOD required by the model may not always accurately represent the physical system, and in this instance the computed BOD data exceeds the observed data.

The solids accumulation rates determined through calibration of the North Jesuit basin are higher that those of any of the other basins. It is speculated that the reason for this is related to the characteristics of the collection system which is largely along the highway. This highway is swept frequently because of the accumulation of solids which occurs on it.

For the most part, the computed concentrations correspond fairly well with the measured concentrations thereby confirming that the accumulation rates input to the model yield a reasonable description of stormwater quality in the basin.

Charter & Harding Water Quality - The computed and observed hydrographs for the discretely sampled events are presented in Figures 2-20 through 2-22. Equipment problems prevented accurate quantification of flow during the events of

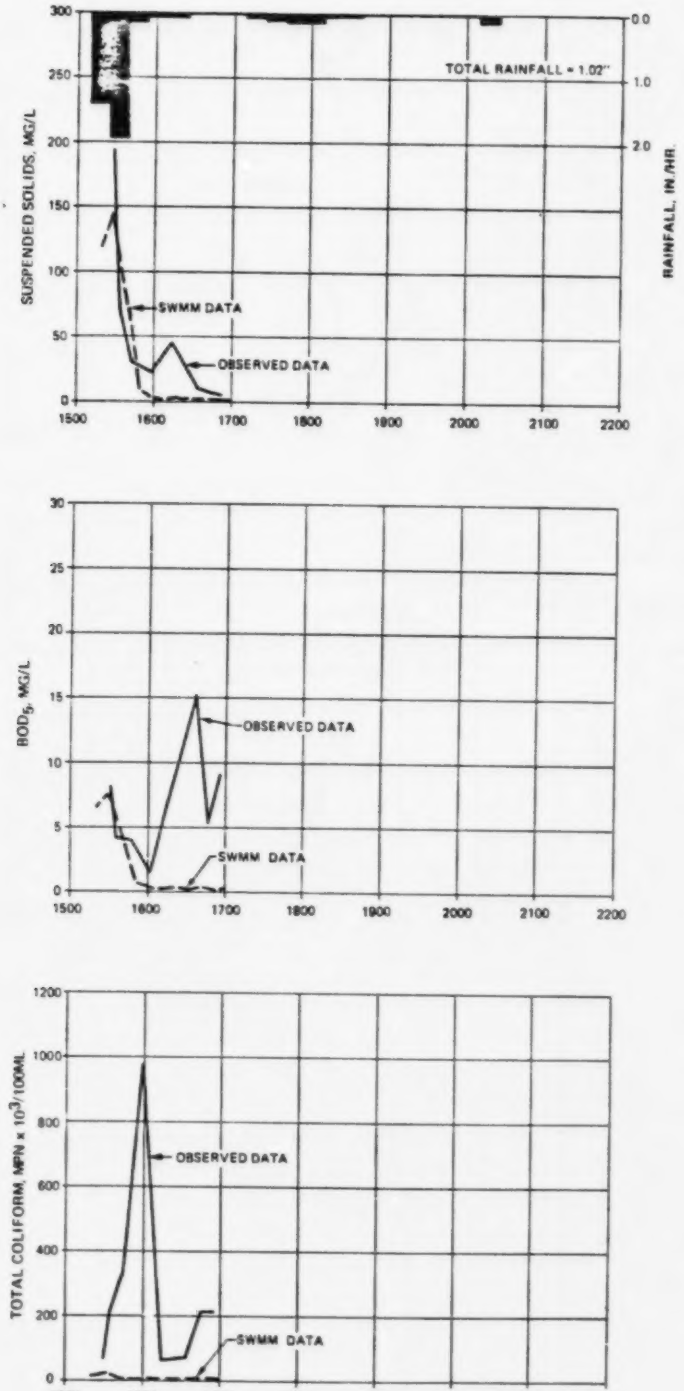


FIG. 2-20 CHARTER & HARDING BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, SEPTEMBER 8, 1982

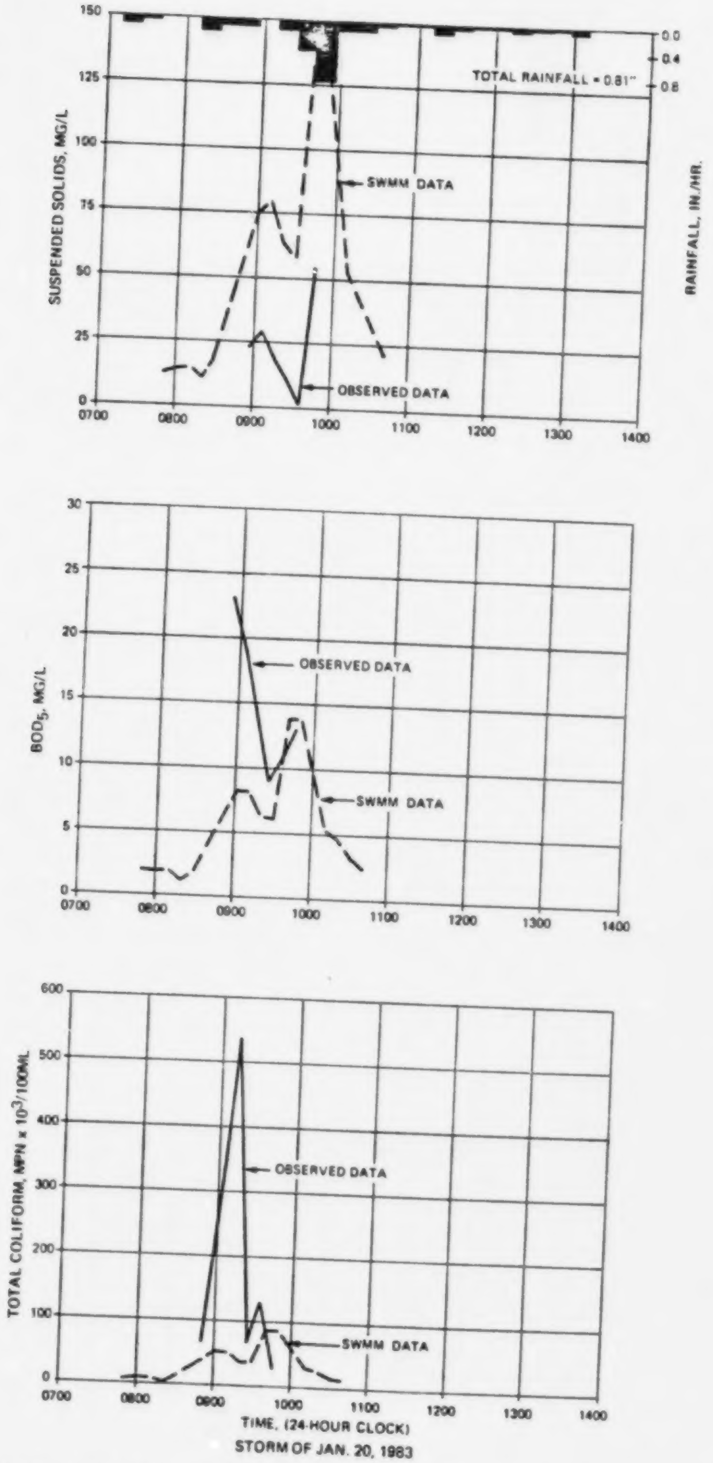


FIG. 2-21 CHARTER & HARDING BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, JANUARY 20, 1983

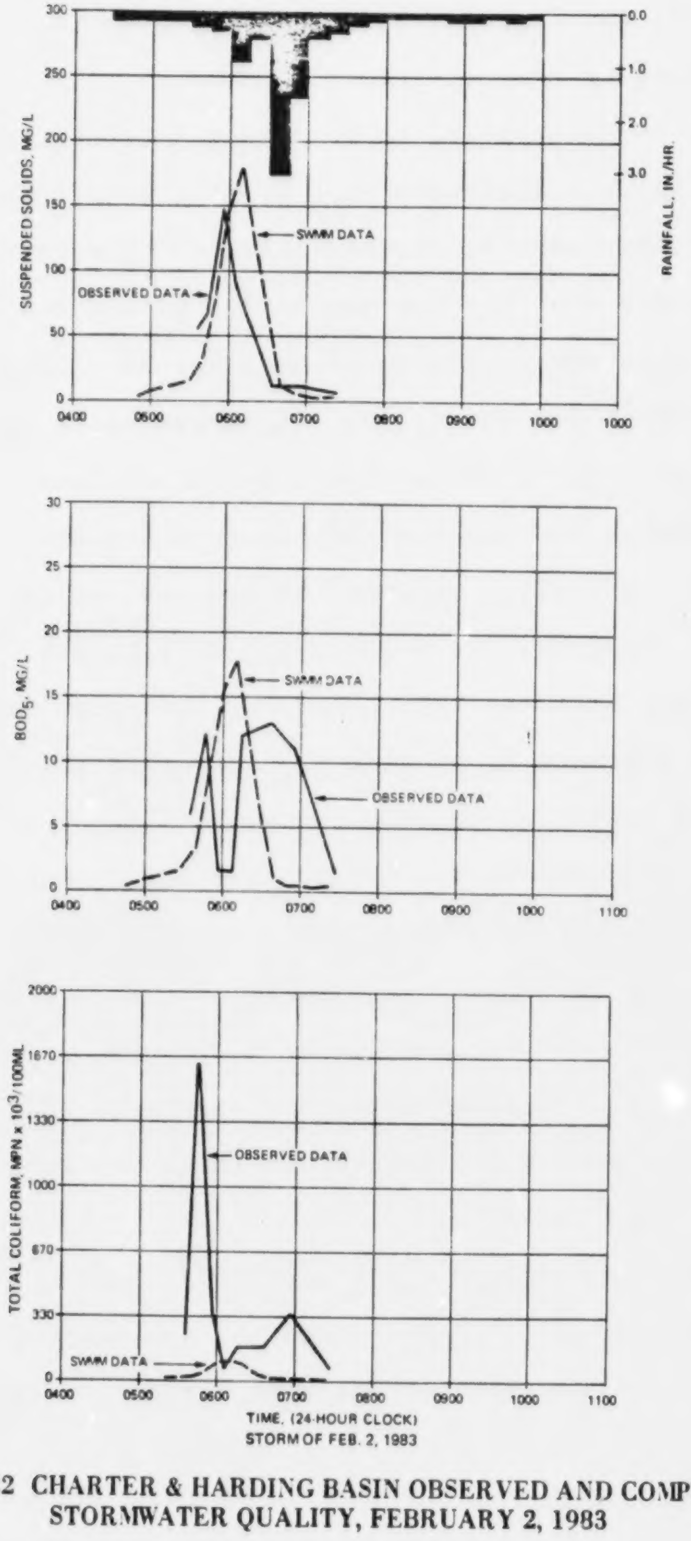


FIG. 2-22 CHARTER & HARDING BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, FEBRUARY 2, 1983

January 20, and February 2, 1983 but the collection of discrete quality samples was not interrupted.

It is interesting to note that coliform concentrations observed in this basin are much greater than those measured in the other four basins. Possibilities of the source of these coliforms are that a cross connection with a sanitary sewer may exist or that the neighborhood contains an unusually large number of pets. Whatever the source, it necessitates the use of a large coliform multiplier within the model to achieve predicted concentrations approaching the observed concentrations.

In the quality calibration process, accumulation rates that best described the storm event of September 8 were about ten times larger that the rates used for the later events. This event was preceded by several days of heavy rainfall and an antecedent condition of only one day was input to SWMM. Therefore computed solids buildup was minor, and consequently little solids were available for washoff. In actuality, the observed data indicates that significant solids were available for washoff. However, many other storm events used for model calibration also had one antecedent dry day. Since in general calibrations were good, this storm event may be somewhat different than the norm.

The storm events which occurred on January 20 and February 2 were both preceded by more than 10 days without rainfall. To obtain a more realistic estimate of the actual solids accumulated and available for washoff, this antecedent condition was reduced to 6 days for input to the model. As shown in Figure 2-21 and 2-22 the predicted outflow solids and BOD concentrations are similar to the observed concentrations.

Norma Park Water Quality - The measured quality data and computed pollutographs for this basin are compared in Figures 2-23 through 2-25. The solids and coliform concentrations measured during the two large events with periods of intense rainfall were significantly larger than those measured during the smaller, less intense storm.

The storm events of September 7 and 8 were each preceded by rainfall on the previous day, and as shown, the computed suspended solids concentrations compare reasonably well with the observed concentrations. The discrepancy between the computed and the observed solids concentrations for the storm of September 8 may reflect the fact that the computed peak flow rate exceeds the observed flow rate.

Thirteen days without rainfall preceded the storm event of September 21; and to model the anticipated plateau of solids accumulation, an antecedent dry period of six days was input to SWMM. It is evident that the SWMM results exceed the observed suspended solids concentrations for this event, thereby indicating that the accumulation computed over the assumed six day period is greater that the actual accumulation. Meteorological conditions or street sweeping may have resulted in the relatively low solids accumulation during this period.

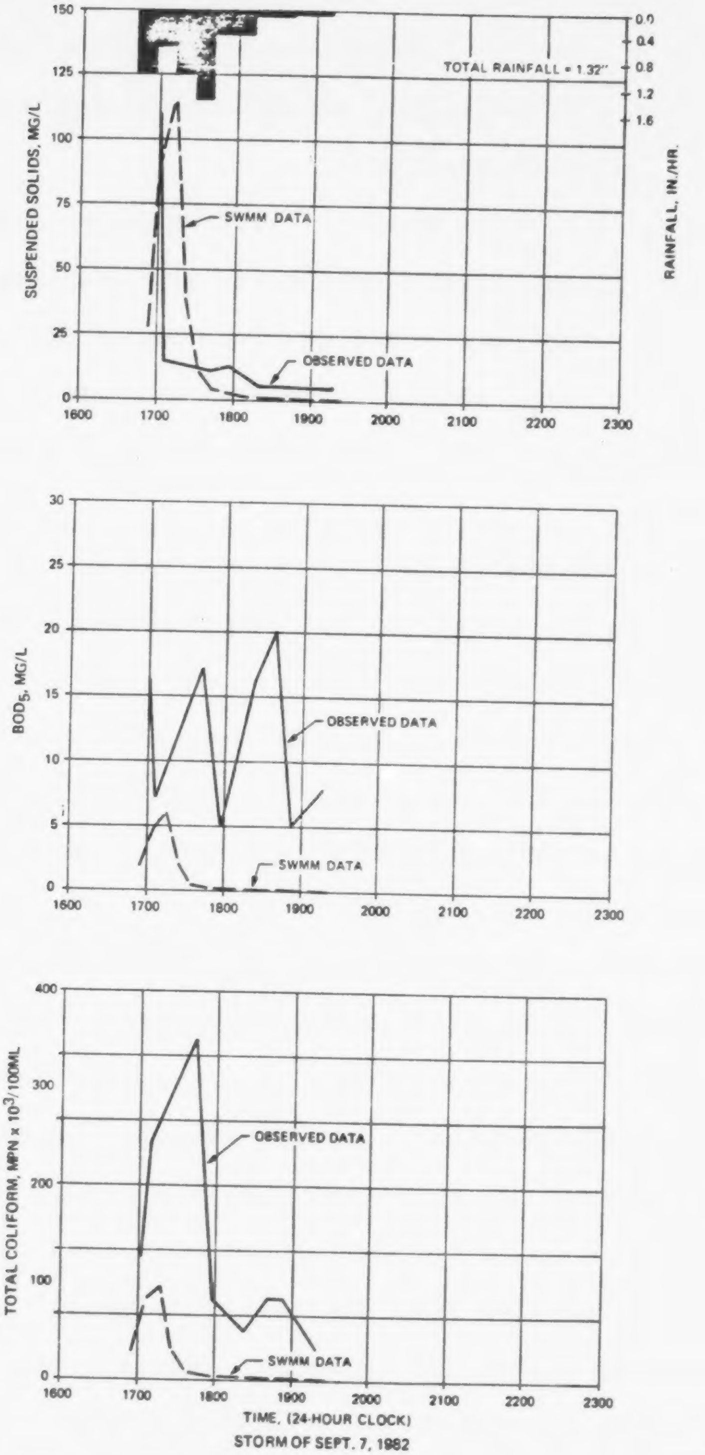


FIG. 2-23 NORMA PARK BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, SEPTEMBER 7, 1982

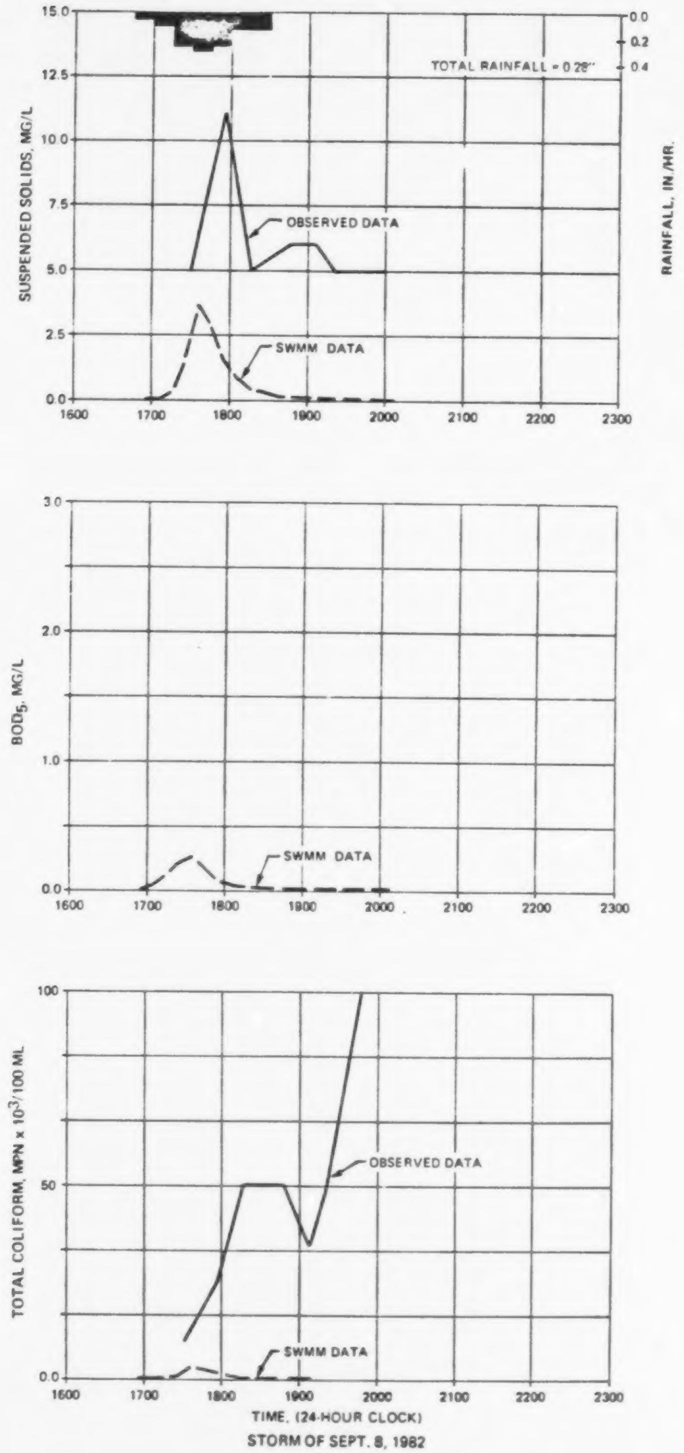


FIG. 2-24 NORMA PARK BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, SEPTEMBER 8, 1982

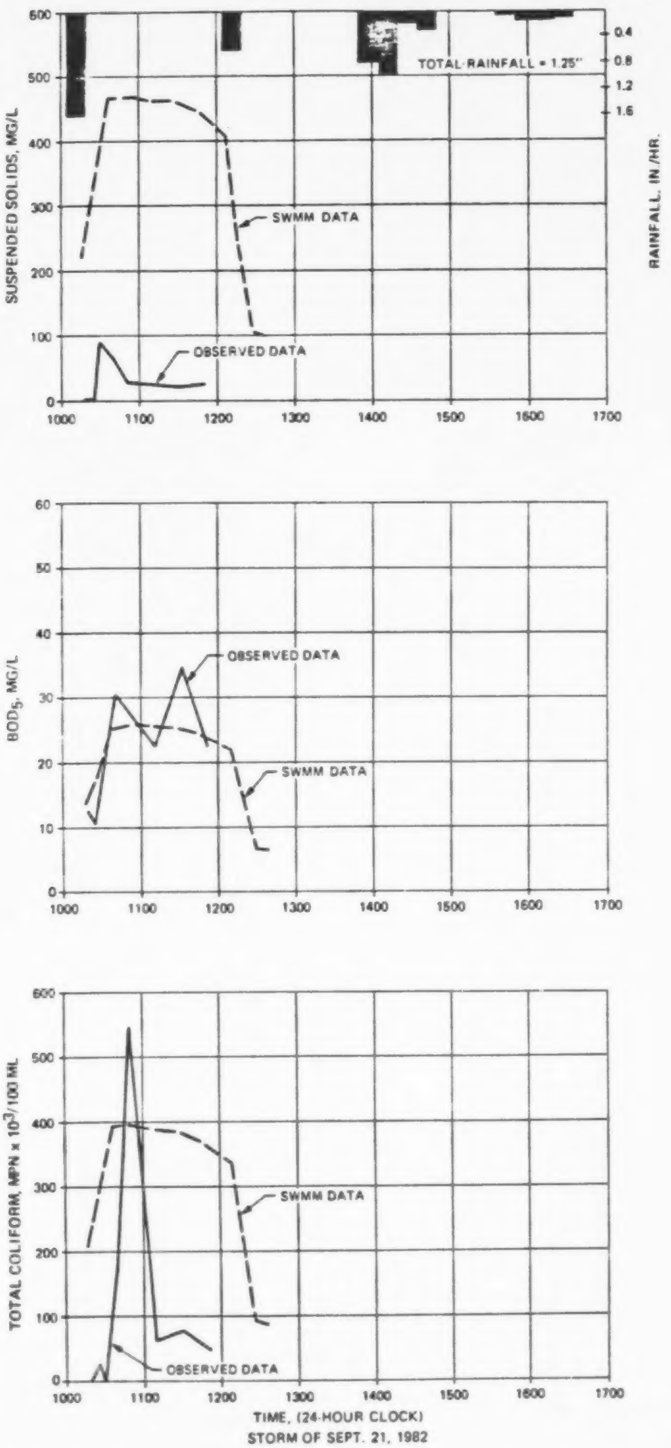


FIG. 2-25 NORMA PARK BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, SEPTEMBER 21, 1982

For the first two events, the computed coliforms are less than the measured concentrations. For the event of September 21, for which computed solids exceed observed solids, the computed coliform concentrations also exceed the measured values. However, if the computed solids accumulation was more realistic for this event, the computed coliform would compare more favorably with measured values. In conjunction with the relatively low solids accumulation rates, for all three storm events a coliform multiplier of 5 is used to improve the coliform calibration. This indicates that the actual coliform availability may be independent of solids buildup.

Summary of Stormwater Quality Modeling. Based upon the above analyses, it is concluded that the major factors affecting stormwater quality in Tampa are the rate of dust and dirt buildup and antecedent condition, which determine the potential pollutant load, and storm intensity, which determines how much of the potential load actually washes off. The washoff process is sensitive to all three of these factors, which are the essential components of pollutant generation. The effects of these factors are described in more detail below.

1. With the exception of the North Jesuit Basin, the best fit, dust and dirt accumulation rates were approximately ten times less than SWMM default values. This indicates that stormwater in Tampa is generally of better quality than in other NURP cities. This is supported by the comparison of Tampa

data with data from other NURP cities, which is presented later in this Chapter.

2. For the storms preceeded by a long dry period, the best fit was usually obtained by reducing the antecedent condition to six days. This suggests that the rate of dust and dirt buildup in Tampa is not constant with time and that after several days the accumulated solids may reach a maximum constant level.
3. The observed BOD concentrations did not exhibit significant variation between events or among the basins. Conversely, the observed coliform concentrations were extremely variable, between events and during events.
4. The largest peak concentrations of suspended solids and coliform were observed during storms with intense rainfall. Peak BOD concentrations did not appear to coincide with peak rainfall intensity. Intense rainfall may permit the washoff of heavier inorganic solids, such as small rocks and stones, which do not contain BOD. This would not normally occur during less intense events. Tampa stormwater BOD is relatively soluble compared with that of other NURP cities. As shown in Table 2-7, the average amount of soluble BOD per pound of dust and dirt used in SWMM calibration is almost an order of magnitude greater than the default values. Tampa stormwater BOD is

primarily not associated with street dirt as indicated by analysis of material collected by street sweepers.

Analysis of Composite Event Data

Modeling of the discrete event data in the previous section was performed to develop an understanding of the within-storm variability of stormwater in Tampa and its causes. A large number of storms were also analyzed on a composite (one sample per storm) basis to develop an understanding of the storm to storm and basin to basin variability of stormwater runoff and its quality. In this section the analysis of these composite data is described.

Data was analyzed for all composite events sampled, as well as the composite values as calculated from the discrete events. The analysis was performed first for the flow data, and then for quality data.

Flow Data. During the study, rainfall and runoff were monitored continuously in each basin. Hydraulic data is available for a total of 51 to 77 events depending on the basin (2,3). For each event the characteristics which were identified included: antecedent dry period, total rainfall, total runoff, storm duration, mean intensity, and mean and peak flow rate. The effect of these storm event characteristics on stormwater runoff quality is analyzed in the next section. In this section, rainfall-runoff volumes are analyzed to determine runoff coefficients for each basin.

A volumetric runoff coefficient (R_c) was calculated based on the percentage of rainfall volume that is converted to runoff as follows:

$$Rc = \frac{\text{runoff volume}}{\text{rainfall volume}}$$

The coefficient R_c takes into account the various factors which influence the generation of runoff flows from rainfall, such as impervious area, infiltration and storage.

The runoff coefficients determined for each basin are presented in Table 2-9, along with the major land use types for each basin. These coefficients represent the mean of all storms during which continuous rainfall and runoff data were collected. Storm events in which data were missing, or in which the calculated value of Rc was greater than 1.1, were not included in this analysis. An Rc of 1.0 means that all rainfall becomes runoff. Clearly Rc cannot physically be more than 1.0. However, to accommodate measurement uncertainty, Rcs of up to 1.1 were included in this analysis.

Based on the information in Table 2-9, the runoff coefficient appears to be a function of the amount of impervious area, except in Norma Park. This basin, although highly impervious, contains large drainage ditches. These ditches, as discussed in the discrete event analysis, are responsible for large reductions in runoff. As mentioned in the previous section on analysis of discrete data, the fraction of imperviousness has a significant impact on the volume of runoff generated in a basin.

TABLE 2-9. RUNOFF COEFFICIENTS AND LAND USE CHARACTERISTICS OF THE MONITORED BASINS

	Total number of events	Number of events analyzed(1)	Average runoff coefficient, Rc	Percent impervious	Percentage Area for Given Land Use Type				
					Industrial/commercial	High density residential	Medium density residential	Low density residential	Institutional/open space
J.L. Young	56	35	0.68	61		100			
Wilder Ditch	77	51	0.45	39	25			55	21
North Jesuit	77	60	0.34	37				48	52
Charter & Harding	56	25	0.35	31				89	11
Norma Park	51	45	0.66	90	91		9		

1. Events with missing data or Rc greater than 1.1 not included in this analysis.

Quality Data. To characterize the quality of runoff in Tampa, and investigate possible interrelationships between measured stormwater quality and other factors, statistical analyses were performed. These analyses included calculating standard statistics for quality composite data from each basin, linear regressions between storm characteristics and stormwater quality, and multiple regressions to attempt to define functional relationships.

The standard statistics used to analyze the quality composite data from each basin and then to compare basin to basin variability were the mean value and standard deviation. Table 2-10 presents these statistics for TSS, BOD_5 and fecal coliform in each basin. The standard deviations listed in Table 2-10 indicate within basin variability in the observed values of approximately the same magnitude as the measurement means. This implies that the true mean values could be significantly different than the measured mean values.

Under these circumstances, only if the measured means from each basin differed by substantially more than the within basin variability could valid relationships be derived establishing pollutant concentrations as a function of the particular basin land uses. However, the measured means are relatively close compared to the within basin variability. This is shown in Table 2-11, which gives means and standard deviations for the pollutants, ascertained from the five mean values for each

TABLE 2-10. OVERALL WATER QUALITY STATISTICS FOR THE MONITORING BASINS

Basin	Sample Size	Water Quality Parameter	Flow Weighted Mean Concentration	Standard Deviation of Measurements
1(1)	15	TSS(6)	42.3	36.9
1	15	BOD_5(6)	13.8	9.3
1	3	Fecal Coliform(7)	15700	4600
2(2)	18	TSS	33.5	22.7
2	18	BOD_5	16.6	15.5
2	2	Fecal Coliform	23300	7000
3(3)	18	TSS	70.9	94.4
3	18	BOD_5	16.7	7.7
3	2	Fecal Coliform	19200	24700
4(4)	15	TSS	29.1	34.2
4	15	BOD_5	10.5	9.0
4	3	Fecal Coliform	40600	18000
5(5)	15	TSS	19.4	15.1
5	14	BOD_5	11.7	6.3
5	3	Fecal Coliform	23000	11700

1. J.L. Young
2. Wilder Ditch
3. North Jesuit
4. Charter & Harding
5. Norma Park
6. mg/l
7. MPN/100ml

TABLE 2-11. STATISTICS OF THE OVERALL VALUES FOR THE FIVE MONITORING BASINS

Water Quality Parameter	Mean	Standard Deviation
TSS(1)	39.0	19.6
BOD(1)	13.9	2.8
Fecal Coliform(2)	24400	9600

1. mg/l
2. MPN/100ml

pollutant in Table 2-10. Thus, this data set suggests that stormwater quality may not be a direct function of land use in Tampa.

Linear regression analysis involves least-squares fitting of a line through a data set composed of independent (causative factors) and dependent (determined factors) variables. A correlation coefficient which ranges from -1.0 to 1.0 is calculated as being representative of the fit of a line to the data. A coefficient of 1.0 indicates perfect correlation with a positive slope and -1.0 indicates perfect correlation with a negative slope. A coefficient of 0.0 indicates that the data are completely uncorrelated.

The independent and dependent variables analyzed are shown in Table 2-12. The analysis was performed first for each basin, and the results are presented in Table 2-13. All relationships in which correlation coefficients greater than 0.5 (or smaller than -0.5) are shown.

TABLE 2-12. STORM CHARACTERISTICS AND POLLUTANTS USED IN LINEAR REGRESSIONS

Abbrv.	Storm Characteristics (independent variable)	Abbrv.	Pollutants (dependent variable)
ANT	Antecedent Condition (days)	BOD	Biochemical Oxygen Demand (mg/l)
DUR	Storm Duration (hours)	COD	Chemical Oxygen Demand (mg/l)
MIN	Maximum 15 Minute Intensity (in/hr)	TSS	Total Suspended Solids (mg/l)
AIN	Avg. Storm Intensity (in/hr)	OP	Ortho-Phosphate (mg/l)
TRV	Total Rainfall Volume (Mgal)	TP	Total Phosphate (mg/l)
QAV	Mean Runoff Flow Rate (mgd)	NIA	Nitrate (mg/l)
QPK	Peak Runoff Flow Rate (mgd)	NII	Nitrite (mg/l)
		TKN	Total Kjeldahl Nitrogen (mg/l)
		NH3	Ammonia (mg/l)
		RNH	Organic Nitrogen (mg/l)
		TN	Total Nitrogen (mg/l)
		CD	Cadmium (ug/l)
		CU	Copper (ug/l)
		PB	Lead (ug/l)
		ZN	Zinc (ug/l)

TABLE 2-13. SUMMARY OF LINEAR REGRESSION ANALYSIS

Basin and Parameter	Correlation Coefficients for Stated Independent Variable							
		ANT	DUR	MIN	AIN	TRV	QAV	QPK
J.L. Young								
BOD_5		-0.52						
Ortho-P	0.91					0.61		
Total P	0.80							
Nitrate	0.63							
TKN	0.58							
Ammonia						0.70		
Total N	0.62							
Cadmium					0.56			
Copper				0.56				
Wilder Ditch								
TSS			0.66	0.52				
Total P			0.63	0.52				
TKN				0.54				
Org. N				0.55				
North Jesuit								
Total P	0.57			0.56				
Nitrite	0.54							
Lead	0.60		0.61				0.60	
Charter & Harding								
COD	0.71							
TSS	0.86							
Ortho-P	0.60							
Nitrate			0.51		0.66			
TKN	0.61							
Org. N	0.55							
Total N	0.50							
Norma Park								
BOD_5		0.67		-0.67		0.52		
COD						0.58		
TSS			0.74		0.70			
Ortho-P			0.81		0.82			
Total P		0.57	0.55					
Nitrite							0.67	
TKN		0.51						
Ammonia		0.68						
Org. N							-0.56	
Total N		0.60			0.50			
Cadmium					0.72			
Copper				-0.55				
Lead			0.53					
Zinc		0.66		-0.63				

As shown in Table 2-13, in only one case was the correlation coefficient above 0.9, and in four cases the coefficients were between 0.8 and 0.9. The coefficients are quite random, and seem to have little dependence on any particular factor. Antecedent condition may be important in the J.L. Young, North Jesuit and Charter & Harding basins. Storm duration may be important in the Norma Park basin and rainfall intensity may be important in the Wilder Ditch basin.

All data collected in the individual basins were subsequently pooled and analyzed as one data set. For the pooled data from all five basins, in no case was there a coefficient of 0.5 or greater (or -0.5 or less). The correlation matrix for the pooled data are shown in Table 2-14. The average coefficient for the antecedent condition is about 0.2, while for all other independent variables it is about 0.1. As determined from the discrete event modeling, adjustment of antecedent condition was necessary to achieve calibration, and therefore has some influence on stormwater quality.

It appears from the results of the linear regression analysis that stormwater quality in Tampa is not a predictable function of storm characteristics. In order to further verify this, multiple linear regression analyses were performed.

The regression equations were based upon use of antecedent condition, rainfall intensity and storm duration as independent

TABLE 2-14. CORRELATION MATRIX FOR POOLED DATA

ANT	DUR	MIN	AIN	TRV	QAV	QPK	
BOD	-.06	-.16	-.03	-.11	.02	.07	.01
COD	.40	-.17	-.03	.11	-.09	-.06	-.01
TSS	.24	-.12	.19	.13	-.05	.07	.08
OP	.36	.01	.21	.18	-.09	-.07	.13
TP	-.03	.01	.06	-.06	-.06	.41	-.04
NIA	.05	.02	.32	.17	.02	-.09	.24
NII	.03	.09	-.10	-.15	-.01	.01	.02
TKN	.38	-.11	.05	.08	-.10	-.02	.01
NH3	.17	-.05	.07	.00	-.10	.03	.03
RNH	.28	-.09	.00	.08	-.06	-.02	-.01
TN	.35	-.11	.02	.08	-.10	-.02	.00
CD	.07	.05	.10	-.01	-.12	-.07	-.07
CU	-.01	.01	.16	.07	-.08	.00	.02
PB	.26	-.17	.14	.14	.12	.12	.19
ZN	.22	-.13	-.03	-.08	-.05	.10	-.01

Note: See Table 2-10 for explanation of variable abbreviations.

variables. The selected dependent variables were TSS and BOD_5, as they were identified as potential problem pollutants during Phase I (12) of the NURP study, and lead, as it had somewhat higher regression coefficients in Table 2-14. The six regression equations equations tested are given in Table 2-15. Cases 1 through 3 are linear, and 4 through 6 are logarithmic.

TABLE 2-15. REGRESSION EQUATIONS TESTED

Case	Equation
1	$TSS = a\ ANT + b\ DUR + c\ AIN + d$
2	$BOD = a\ ANT + b\ DUR + c\ AIN + d$
3	$PB = a\ ANT + b\ DUR + c\ AIN + d$
4	$TSS = ANT^{a} \cdot DUR^{b} \cdot AIN^{c}\ d$
5	$BOD = ANT^{a} \cdot DUR^{b} \cdot AIN^{c}\ d$
6	$PB = ANT^{a} \cdot DUR^{b} \cdot AIN^{c}\ d$

A summary of the multiple regression analysis is presented in Table 2-16. For over 70 percent of the cases, the regressions were statistically insignificant. The cases which do exhibit statistical significance are randomly distributed throughout the basins.

Based upon the linear and multiple regressions performed, it can be concluded that the derivation of functional relationships between quality parameters and storm characteristics cannot be statistically justified. Therefore, when area-wide flows and loads are developed in Chapter 4, the total data set regardless of storm event characteristics will be utilized.

Monitor Other NURP Projects (Task II.11)

As a further step in characterizing stormwater runoff in Tampa, the stormwater quality data collected were compared with nationwide NURP data (13). As shown in Table 2-17, for all

TABLE 2-16. SUMMARY OF MULTIPLE REGRESSION ANALYSIS

Case	Significance Basin	Correlation Level(1)	Coefficient
TSS - Linear	Overall	INS	0.27
	J.L. Young	INS	0.38
	Wilder Ditch	INS	0.52
	N. Jesuit	INS	0.29
	Charter & Harding	***	0.89
	Norma Park	INS	0.54
BOD - Linear	Overall	INS	0.24
	J.L. Young	INS	0.41
	Wilder Ditch	INS	0.25
	N. Jesuit	INS	0.45
	Charter & Harding	INS	0.13
	Norma Park	***	0.84
Lead - Linear	Overall	**	0.31
	J.L. Young	INS	0.51
	Wilder Ditch	INS	0.48
	N. Jesuit	***	0.71
	Charter & Harding	INS	0.43
	Norma Park	*	0.69
TSS - Log	Overall	*	0.30
	J.L. Young	INS	0.45
	Wilder Ditch	**	0.67
	N. Jesuit	INS	0.45
	Charter & Harding	INS	0.39
	Norma Park	INS	0.66
BOD - Log	Overall	INS	0.17
	J.L. Young	INS	0.53
	Wilder Ditch	INS	0.27
	N. Jesuit	INS	0.48
	Charter & Harding	INS	0.24
	Norma Park	***	0.90
Lead - Log	Overall	INS	0.24
	J.L. Young	**	0.72
	Wilder Ditch	INS	0.36
	N. Jesuit	*	0.63
	Charter & Harding	INS	0.44
	Norma Park	INS	0.48

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1. INS = Insignificant, * = 90 percent, ** = 95 percent, *** = 99 percent.

TABLE 2-17. COMPARISON OF TAMPA AND NATIONWIDE STORMWATER QUALITY DATA

Parameter	(Unit)	Mean concentration of pooled runoff data - Tampa	Mean concentration of pooled nationwide runoff data[1]	(No. of events)
BOD_5	(mg/l)	14	8	
COD	(mg/l)	51 ?	71	(709)
TSS	(mg/l)	40	108	(790)
Ortho-P	(mg/l)	0.14		
Total-P	(mg/l)	1.38	0.29	(737)
Nitrate	(mg/l)	1.89		
Nitrite	(mg/l)	0.02		
TKN	(mg/l)	1.24	1.5 NO2 NO3	(1172)
Ammonia-N	(mg/l)	0.38		
Organic-N	(mg/l)	0.91		
Total-N	(mg/l)	1.66		
Cadmium	(ug/l)	2.49	18.2	(3,564) ???
Copper	(ug/l)	49	33	(790)
Lead	(ug/l)	57	142	(1249)
Zinc	(ug/l)	59	134	(962)
Arsenic	(ug/l)	9		
Barium	(ug/l)	35		
Chromium	(ug/l)	8.4	33.4	(3,130)
Iron	(ug/l)	545		
Manganese	(ug/l)	14		
Mercury	(ug/l)	6.9		
Nickel	(ug/l)	13	24.4	(804)

---

1. NURP Nationwide report, first draft, September 28, 1982 (3).

of the parameters except BOD_5, copper, and phosphorus, Tampa runoff has lower concentrations. Total suspended solids, cadmium, lead, chromium and zinc are substantially lower than the nationwide averages. This supports the findings of the stormwater modeling discussed earlier in this Chapter.

Sources of Stormwater Constituents

The stormwater quality in Tampa can be derived either from the rainfall itself, or from the land surface by washoff. In order to determine from which source the pollutants are derived, a comparison of rainfall quality (14) and stormwater quality was made. This is presented in Table 2-18.

Table 2-18 suggests that stormwater copper and zinc comes primarily from rainfall. Stormwater TSS, total-P, lead and iron clearly derive significantly from surface buildup. For the other stormwater constituents, both sources may contribute substantially.

Summary

In order to characterize the stormwater runoff quantity and quality in Tampa, the following analyses have been conducted on the available data set:

1. Single-event hydraulic and water quality modeling of discrete data using SWMM.
2. Statistical analysis of hydraulic and water quality composite data.

TABLE 2-18. SUMMARY OF MEAN RAINFALL QUALITY CONCENTRATIONS

Parameter	Mean concentration in pooled run- (Unit)	Range of mean concentrations of rainfall off data	quality (1)
BOD_5	(mg/l)	14	2 - 4
COD	(mg/l)	51	13 - 20
TSS	(mg/l)	40	4 - 11
Ortho-P	(mg/l)	0.14	0.04 - 0.19
Total-P	(mg/l)	1.38	0.10 - 0.23
Nitrate	(mg/l)	1.89	0.19 - 0.87
Nitrite	(mg/l)	0.02	0.004 - 0.014
TKN	(mg/l)	1.24	0.34 - 0.55
Ammonia-N	(mg/l)	0.38	<0.10 - 0.12
Organic-N	(mg/l)	0.91	0.34 - 0.53
Total N	(mg/l)	1.66	0.51 - 1.23
Cadmium	(ug/l)	2.49	0.55 - 1.5
Copper	(ug/l)	49	32 - 245
Lead	(ug/l)	57	3.8 - 7.3
Zinc	(ug/l)	59	92 - 189
Arsenic	(ug/l)	9	5 - 6
Barium	(ug/l)	35	<10 - 11
Chromium	(ug/l)	8.4	2.1 - 6.8
Iron	(ug/l)	545	9 - 45
Manganese	(ug/l)	14	3 - 8
Mercury	(ug/l)	6.9	1 - 2.8
Nickel	(ug/l)	13	1.5 - 4

1. Task II.3 Precipitation Quantity and Quality (14).

3. Comparison of Tampa data to nationwide data.

Based on these analyses, the following conclusions are drawn:

1. Factors of significance in runoff generation include rainfall, infiltration, basin and transport system characteristics including percent imperviousness, ditch systems and in-system storage.
2. Factors which appear to impact the observed runoff quality include: dust and dirt accumulation, antecedent condition, and rainfall intensity.

3. Stormwater quality is not consistently correlated with particular storm characteristics, and the derivation of regression relationships to predict stormwater quality is not statistically justified. (why?) wrong type of regression analysis

4. Stormwater quality in Tampa is generally better than the quality of nationwide data.

5. Rainfall in Tampa is an important source of some nutrients and metals. √

Try regression of the means.

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CHAPTER 3

ANALYSIS OF STORMWATER CONTROLS

General

Four types of stormwater controls common in the City of Tampa were tested for runoff and pollutant removal capacity. Tests were performed on open bottom inlets, drain trenches and ditches to determine their capability to reduce hydraulic load to storm sewers. A field program was designed and implemented to measure stormwater hydraulics and pollutant loads flowing to and from detention ponds in order to assess their capability for load removal. Details regarding the selection of control testing sites and methods of data collection have been presented elsewhere (1, 2, 3).

Open bottom inlets, drain trenches and ditches were tested in the period from October through December, 1982. Some additional tests were performed in the spring of 1983. Open bottom inlets and drain trenches were tested during both dry and wet weather. The dry weather tests followed a three-week period during which the average daily rainfall was 0.02 inches/day. The wet weather tests followed a relatively wet three-week period with an average daily rainfall of 0.12 inches/day.

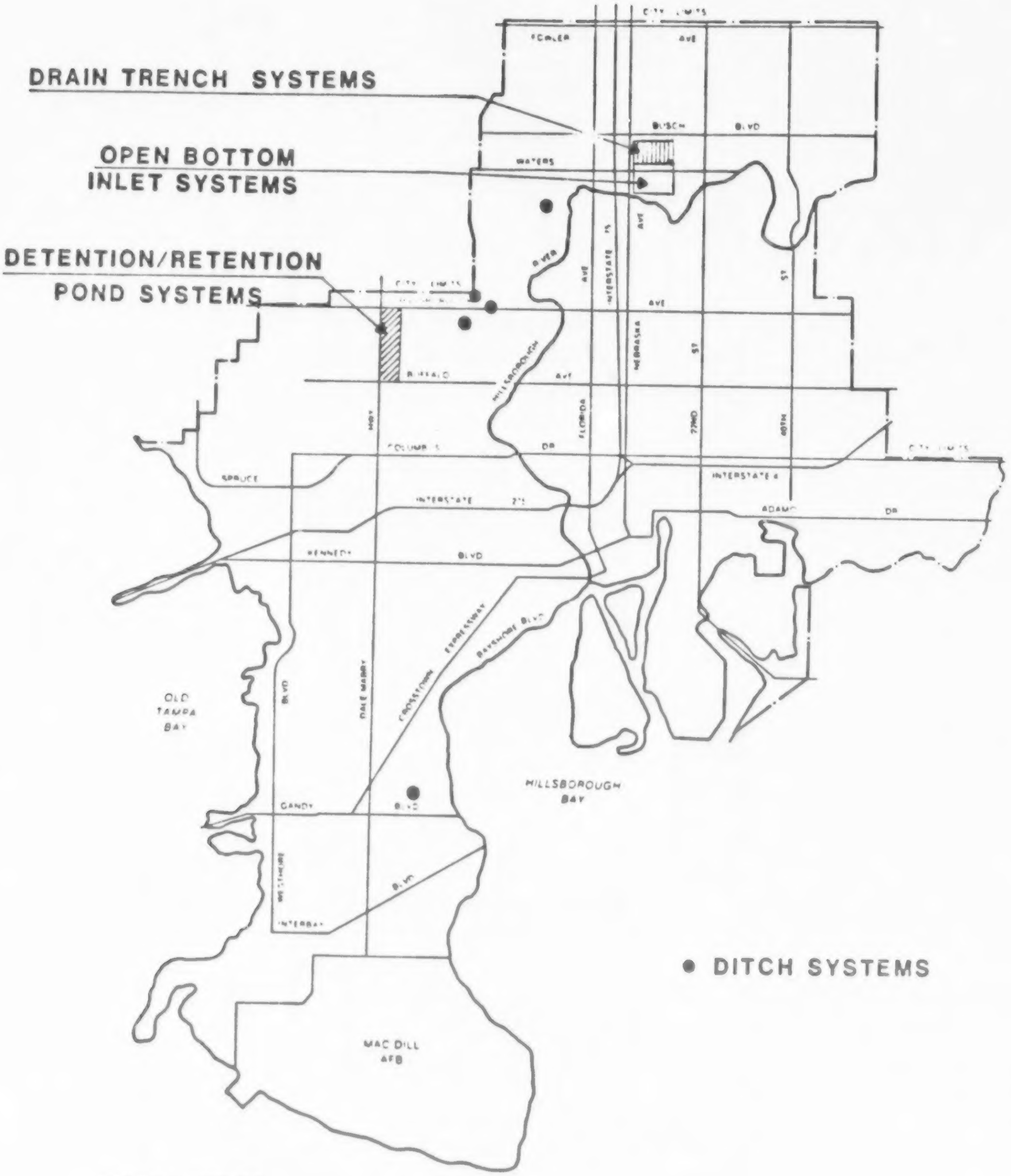
Ditch systems were tested only in dry weather conditions with the exception of one additional test performed in the spring of 1983. The field program for detention pond testing was carried out from May, 1982 through February, 1983.

The test sites for the four stormwater control tests are shown in Figure 3-1. Open bottom inlet systems and drain trenches are located in the Sulphur Springs area just north of the Hillsborough River. The ditch systems and the detention pond site are located west of the river in the vicinity of Hillsborough Avenue.

The purpose of this Chapter is to present and analyze the tests and resulting hydraulic and pollutant removal capabilities of these four stormwater controls. Each type of control will be discussed separately in terms of the tests performed, the analysis of the results, and the conclusions regarding their ability to remove hydraulic flows. Pollutant concentrations were measured only at the detention pond sites; therefore, pollutant removal capabilities of the other three controls will be assessed based on removal of hydraulic flows only.

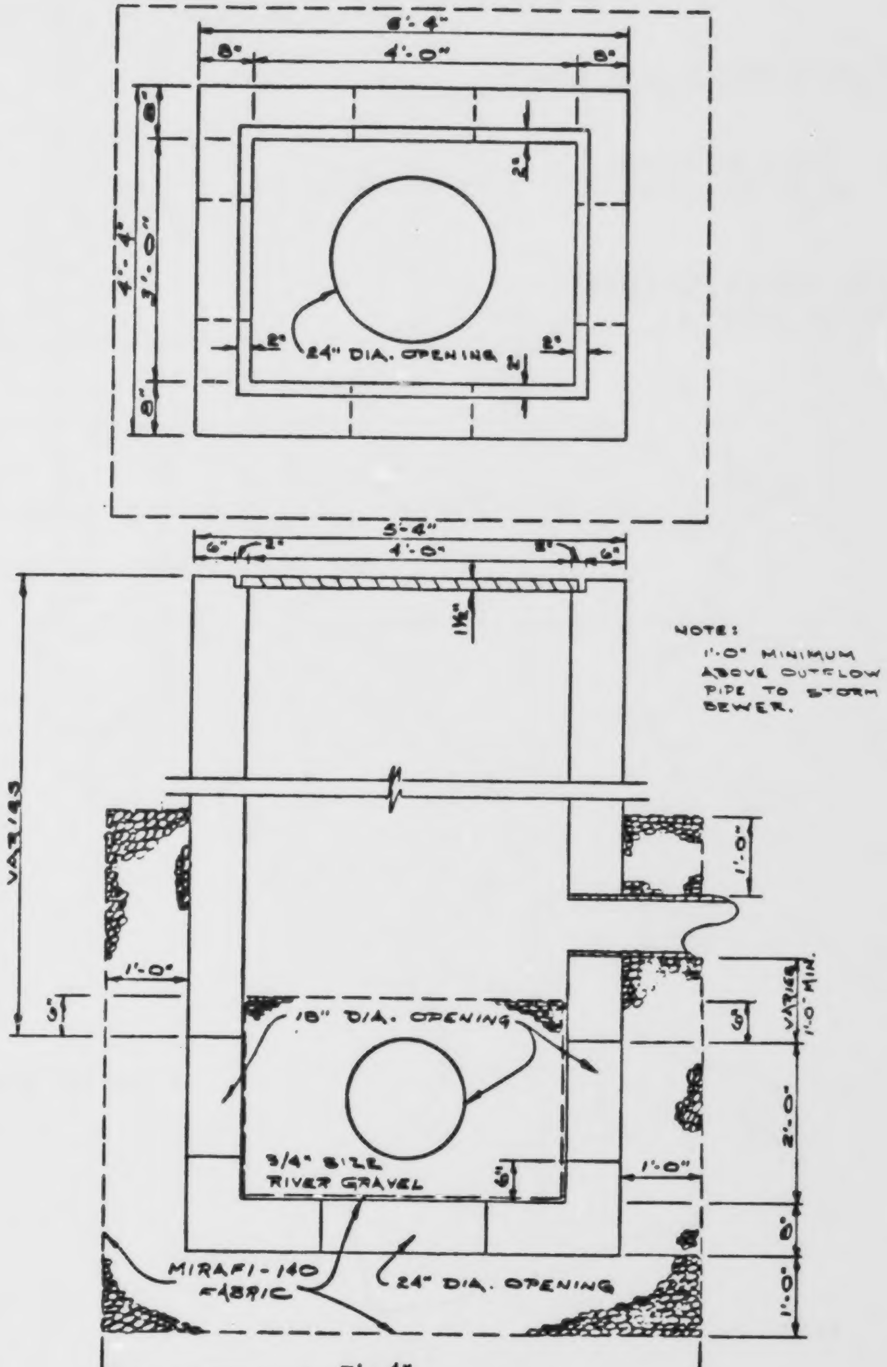
Open Botton Inlets

Description and Summary of Tests. Open bottom inlets (OBI) are similar to standard catch basins. They are designed not only to collect runoff, but also to store a certain portion of the runoff and allow it to infiltrate to the surrounding soil. The OBI's concrete box-like structure is installed on a bed of river gravel and surrounded by a gravel pack. A typical OBI is shown in Figure 3-2. There is also a gravel pack inside the concrete box below the outflow pipe that leads to the storm sewer. The approximate storage volume within the concrete box is


DRAIN TRENCH SYSTEMS
OPEN BOTTOM INLET SYSTEMS
DETENTION/RETENTION POND SYSTEMS
● DITCH SYSTEMS
CITY LIMITS
FOWLER AVE
BUSCH BLVD
WATERS
HILLSBOROUGH RIVER
FLORIDA AVE
INTERSTATE 75
NEBRASKA AVE
22ND ST
40TH ST
BUFFALO AVE
COLUMBUS DR
SPRUCE
INTERSTATE 275
INTERSTATE 4
ADAMO DR
KENNEDY BLVD
CROSSTOWN EXPRESSWAY
BAYSHORE BLVD
OLD TAMPA BAY
HILLSBOROUGH BAY
GANDY BLVD
WESTSHORE BLVD
DALE MABRY
INTERBAY BLVD
MAC DILL AFB

SOURCE: TASK II.8

FIG. 3-1 SITE LOCATION MAP FOR STORMWATER CONTROL TESTS


6'-4"
8"
4'-0"
8"
4'-4"
3'-0"
8"
8"
2"
24" DIA. OPENING
2"
2"
2"
5'-4"
6"
2"
4'-0"
8"
6"
1½"
VARIES
1'-0"
3"
18" DIA. OPENING
3/4" SIZE RIVER GRAVEL
3"
1'-0"
VARIES 1'-0" MIN.
3"
2'-0"
8"
1'-0"
1'-0"
MIRAFI-140 FABRIC
24" DIA. OPENING
7'-4"

NOTE: 1'-0" MINIMUM ABOVE OUTFLOW PIPE TO STORM SEWER.

SOURCE: CITY OF TAMPA

FIG. 3-2 TYPICAL DESIGN OF OPEN BOTTOM INLETS

15 cubic feet and about 55 cubic feet of void volume are available in the gravel pack surrounding the box.

The OBI test sites are located in the Waters Avenue Storm Sewer Basin. This area is almost fully developed, and is predominantly residential with some industrial land uses. The soils in this basin are reported to be well drained. Tests on three different OBI's were performed on October 12 and November 19, 1982. Table 3-1 presents a summary of these tests. Further details may be found in the Task II.7 and II.8 reports (1,2) of the NURP study.

Analysis of Results. Several observations can be made from Table 3-1. First, the length of time from the start of the test to the beginning of outflow is very short for all tests. Although there is approximately 70 cubic feet of available storage space in and around the OBI's the largest volume retained before outflow began was 14 cubic feet at OB-1 on Nov 19, 1982. This volume barely filled the available void space (15 cubic feet) within the concrete box itself. All other tests retained less volume before discharge began. This indicates that the OBI's were initially saturated with water or that debris clogged the openings to the gravel pack around the concrete box, thereby preventing significant water movement into the gravel voids.

Table 3-2 compares the depths below surface of the gravel packs for the three OBI's to the groundwater levels at the nearest available well during the testing period. This Table indicates that the groundwater level was well below the gravel

TABLE 3-1. SUMMARY OF CONTROL TESTS PERFORMED ON OPEN BOTTOM INLETS

Test	Date	Antecedent conditions	Inflow(1) (cfm)	Time from start of test when outflow begins (min.)	Final outflow(1) (cfm)	Length of test(1) (min.)	Available storage (ft^3)	Total volume removed during test (ft^3)
OB-1	10/12/82	Wet	1.4	8	1.3	11	70	10.7
	11/19/82	Dry	7.0	2	4.7	5	70	21.0
OB-2A	10/12/82	Wet	7.3	1.5	7.4	4	52 (2)	10.9
	11/19/82	Dry	2.2	4	2.7	6	59 (2)	8.7
OB-2B	10/12/82	Wet	5.8	0.5	5.7	2.75	67 (2)	3.2
	11/19/82	Dry	3.8	2	3.4	6	70	8.8

1. Source: Task II.8.
2. Storage reduced due to groundwater level above OBI bottom (See Table 3-2).

TABLE 3-2. GROUNDWATER LEVELS DURING CONTROL TESTING OF OPEN BOTTOM INLETS (1)

Test site	Distance from well to test site (mi)	Depth of OBI gravel pack below surface (ft)	Date	Groundwater level below surface (ft)	Extent of OBI below groundwater table (ft.)
OB-1	0.25	8.67	10/12/82	11.30	0
			11/9/82	12.60	0
OB-2A	0.30	9.57	10/12/82	7.96	1.61
			11/19/82	8.76	0.81
OB-2B	0.30	8.15	10/12/82	7.96	0.19
			11/19/82	8.76	0

1. Source: City of Tampa

pack for OB-1 for both tests. Test site OB-2A, the deepest control, was partly saturated with groundwater but theoretically had more storage capacity than the test results indicate. Test site OB-2B was marginally affected by groundwater and should have retained more runoff than was observed.

The values of total volume of inflow retained during each test as presented in Table 3-1 indicate that in all but one case the runoff did not enter the gravel pack surrounding the OBI's. The dry weather test for OB-1 on November 19, 1982 did begin filling the storage volume around the OBI. However, it appears that this test was terminated before a steady-state outflow was reached. It is difficult, therefore to make conclusions regarding the performance of this OBI during the test.

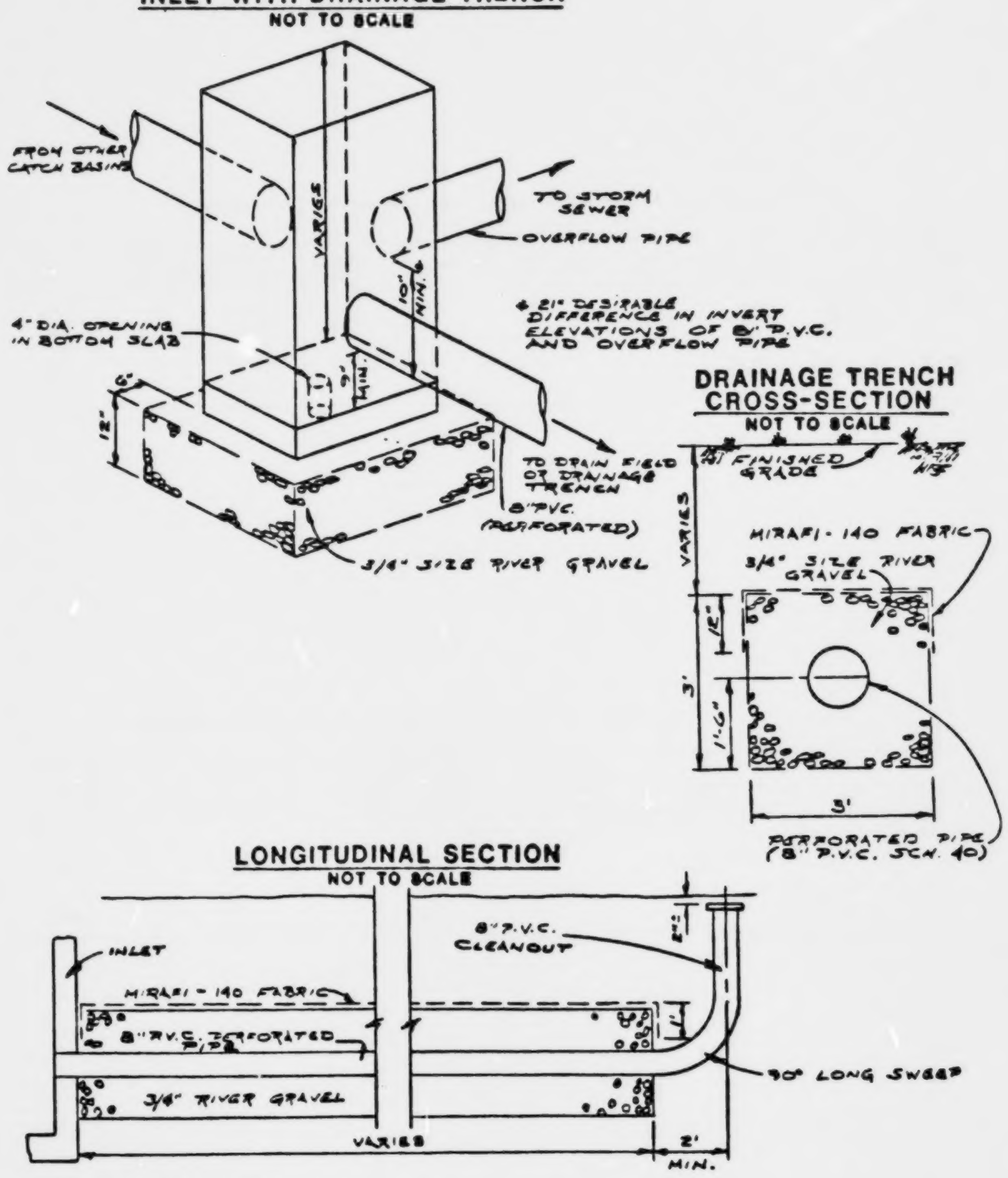
Summary. The control tests performed on open bottom inlets indicate that, in general, the available void space in the gravel pack within and around the concrete box is not utilized for storage of runoff.

It appears that open bottom inlets, though easy to construct, are not very effective in storing stormwater runoff. Measured groundwater levels close to the test sites indicate that the soil around the OBI's was not saturated during most of the tests. Therefore, groundwater cannot be the major obstacle to their performance. It may be reasonable to assume that the filter fabric placed above the gravel pack (Figure 3-2) could become clogged with silt and debris. This would prevent significant runoff volume from seeping into the gravel and may be the major reason why the OBI's demonstrated limited runoff removal capacity. It is recommended that two additional tests be conducted on the OBI's with the filter fabric removed. The first test should be conducted immediately after the removal of the fabric and the second several weeks later to observe any change in the OBI's performance.

Drain Trenches

Description and Summary of Tests. Drain trenches or drainfields are underground trenches that have been excavated and filled with river gravel to allow for storage and infiltration of stormwater runoff entering via a perforated pipe running the length of the trench. Inlets for drain trenches are slightly modified curb inlets that are usually connected to one or more conventional catch basins. Figure 3-3 presents a typical drain trench inlet and the standard design of a drain trench.

Two drain trench test sites were selected in the basin served by the 9th Street Storm Sewer Extension just north of the OBI test sites. Tests on the drain trenches were performed on October 12 and November 19, 1982. A test of longer duration was requested by M&E in 1983 and performed by the City of Tampa on April 28, 1983 after a wet 3-week period with average daily rainfall of 0.11 inches. Test results are summarized in Tables 3-3 and 3-4. More detailed information on the 1982 tests is presented in the Task II.8 report (2). An inflow-outflow hydrograph for the additional 1983 test is presented in Figure 3-4.


INLET WITH DRAINAGE TRENCH
NOT TO SCALE
FROM OTHER CATCH BASINS
TO STORM SEWER
OVERFLOW PIPE
VARIES
10" MIN.
* 21" DESIRABLE DIFFERENCE IN INVERT ELEVATIONS OF 8" P.V.C. AND OVERFLOW PIPE
4" DIA. OPENING IN BOTTOM SLAB
6"
12"
3" MIN.
TO DRAIN FIELD OF DRAINAGE TRENCH
8" PVC. (PERFORATED)
3/4" SIZE RIVER GRAVEL
DRAINAGE TRENCH CROSS-SECTION
NOT TO SCALE
FINISHED GRADE
VARIES
MIRAFI - 140 FABRIC
3/4" SIZE RIVER GRAVEL
12"
3'
1'-6"
3'
PERFORATED PIPE (8" P.V.C. SCH. 40)
LONGITUDINAL SECTION
NOT TO SCALE
INLET
8" P.V.C. CLEANOUT
2"±
MIRAFI - 140 FABRIC
1'
8" P.V.C. PERFORATED PIPE
90° LONG SWEEP
3/4" RIVER GRAVEL
VARIES
2' MIN.

SOURCE: CITY OF TAMPA

FIG. 3-3 TYPICAL DESIGN OF DRAIN TRENCHES

TABLE 3-3. SUMMARY OF CONTROL TESTS PERFORMED ON DRAIN TRENCHES (1)

Test site	Date	Antecedent conditions	Inflow (cfm)	Time from start of test when outflow began (min)	Flow rate at end of test			Length of test (min)
					Outflow (cfm)	Removal		
						(cfm)	(% of inflow)	
DT-1	10/12/82	Wet	4.9	18	2.3	2.6	53	27.5
	11/18/82	Dry	21.2	14	15.2	6.0	28	83.0
	4/28/83	Wet	7.3	53	4.7	2.6	36	240.0
DT-2	10/12/82	Wet	21.8	1	15.8	6.0	28	5.0
	11/18/82	Dry	9.7	23	6.2	3.5	35	73.0

1. Source: Task II.8.

TABLE 3-4. ADDITIONAL DATA ON CONTROL TESTS FOR DRAIN TRENCHES

Test site	Date	Inflow (cfm)	Available storage volume(1) (ft³)	Volume of inflow in drain trench when outflow begins (ft³)	Total volume of inflow over entire test period (ft³)	Total volume of water removed (ft³)
DT-1	10/12/82	4.9	274	88	135	118
	11/18/82	21.2		297	1,760	915
	4/28/82	7.3		386	1,750	1,044
DT-2	10/12/82	21.8	247	22	109	60
	11/18/82	9.7		222	705	426

1. Source: Task II.8.

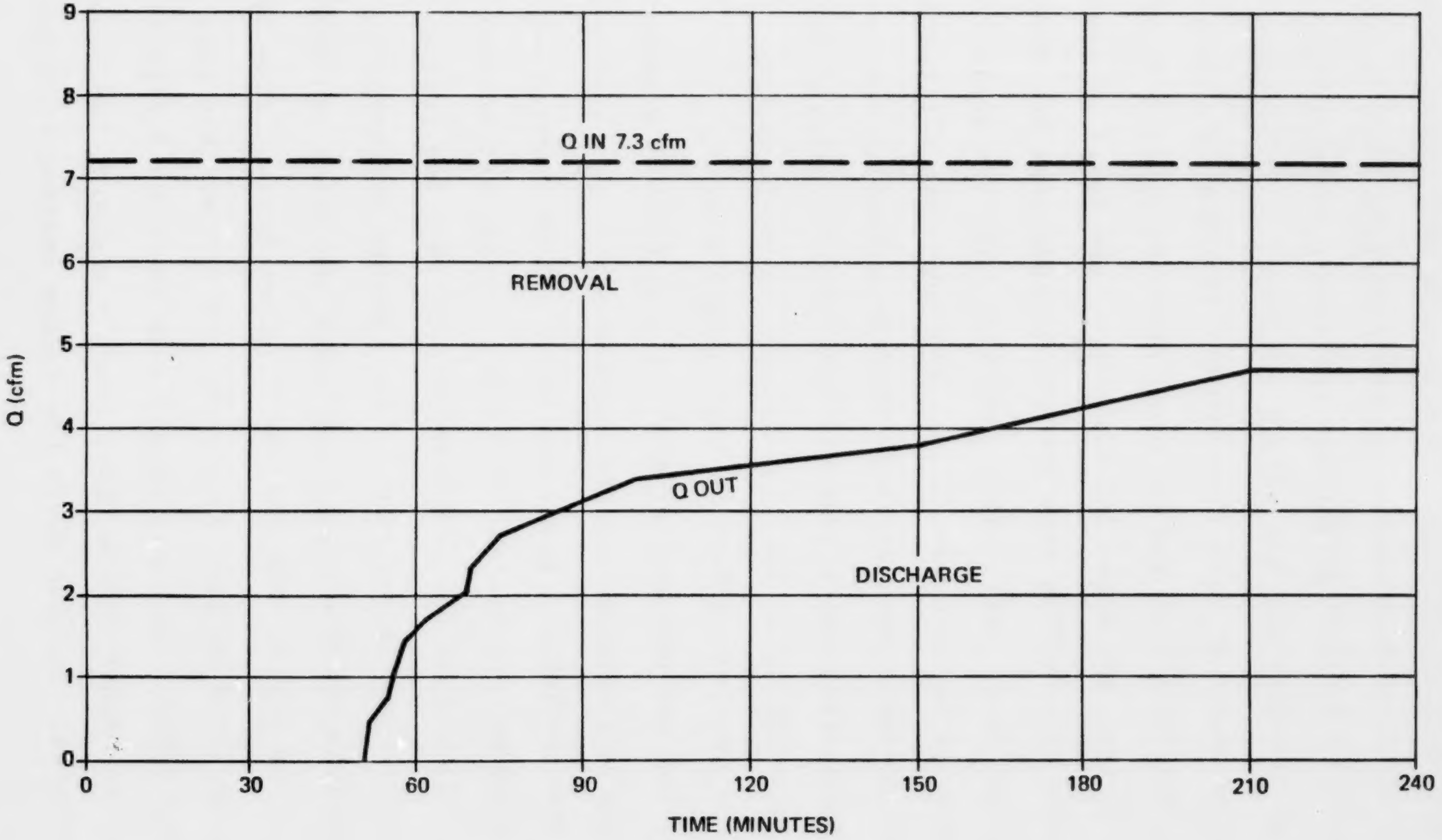


FIG. 3-4 INFLOW-OUTFLOW RESPONSE FOR APRIL 28, 1983 CONTROL TEST AT SITE DT-1

Analysis of Results. In assessing the hydraulic removal capacity of stormwater controls, the following factors must all be taken into consideration: the inflow and outflow rates, the length of the test and the volume retained. For example, the two wet weather tests at site DT-1 exhibit the same removal rate of 2.6 cfm, but this flow was observed after 28 minutes in the October 12 test and after 240 minutes in the April 28 test. Furthermore, the inflow rate for the October test was much lower than for the April test. This indicates that the drain trench performance in the October 12 test was much less efficient. Table 3-4 presents the available storage volume in the respective drain trench sites and volumetric flow balance results for the tests. This indicates that the October 12 drain trench performance was less effective than on April 28.

During the dry weather test at site DT-1 on November 18, less volume was retained before outflow began than during the wet weather test in April. Also, the total volume removed was lower for the November dry weather test than for the wet weather test in April. This does not necessarily mean that dry weather drain trench performance is inferior to wet weather performance. Though the total inflow volume for these two tests was nearly identical, the inflow rate for the November test at site DT-1 was nearly three times as high as that of the wet weather test in April. The drain trench system probably had more difficulty absorbing the higher inflow rate, thereby yielding the lower removal volumes. In both tests the volume removed exceeded the available storage volume, indicating that soil absorption of the runoff did occur around the drain trench. In addition, the outflow rates indicate that significant volumes were still being removed, even after long periods of time.

The volumes in Table 3-4 for the October test at site DT-2 indicate problems with the drain trench performance. Only 22 cubic feet of runoff was retained before outflow was observed, and the total volume removed during the test was less than the available storage volume in the drain trench. For this test, the high inflow rate (hydraulic loading rate) may also have exceeded the absorption capacity of the trench. The short duration of this test raises the question of whether the outflow truly reached steady-state. It is not known, had the test been continued, how much additional flow volume may have been removed.

At site DT-2, the wet weather removal rate is 6.0 cubic feet per minutes (cfm) while that for the dry weather test is only 3.5 cfm (Table 3-3). However, as shown in Table 3-3, the wet weather test at site DT-2 was terminated after 5 minutes as compared to a test length of 73 minutes for the dry weather test. Furthermore, a removal rate of 6.0 cfm at this site is only 28 percent of the inflow rate whereas that for dry weather testing (3.4 cfm) is 35 percent of the inflow. Table 3-4 indicates that the dry weather test at site DT-2 produced good results. Almost the entire storage volume was filled before outflow was observed, and infiltration to the soil around the trench did occur.

The groundwater levels measured near the test sites during the test periods are presented in Table 3-5. These indicate that there was no interference of groundwater in the drain trenches during the control tests.

Equivalent Rainfall Depths. In order to observe the performance of the drain trenches in relation to rainfall depth, an equivalent rainfall depth was calculated for each test. This was done by estimating the rainfall intensity utilizing a modified Rational Method formula:

$$i = Q/ca$$

where i = rainfall intensity (in/hr)

Q = inflow rate (cfs) for test

c = runoff coefficient based on land use

a = drainage area (acres) served by stormwater control

Since the inflow rate for the drain trenches was constant over the test period the calculated rainfall intensity was also assumed to be constant to calculate the rainfall depth. The drainage areas for the two drain trench sites were provided in the Task II.8 report (2). The land use in this basin is low-density residential with 29 percent impervious area. A runoff coefficient of 0.38 was calculated using the equation for R_c as presented in Chapter 4. Table 3-6 presents the equivalent rainfall depths which show that drain trench tests were indicative of less than 0.1 to more than 1 inch of rainfall. Under this variety of conditions, similar percent removals of flow were obtained.

TABLE 3-5. GROUNDWATER LEVELS DURING CONTROL TESTING OF DRAIN TRENCHES [1]

Test site	Distance of well from test site (mi)	Depth of drain trench gravel pack below surface (ft)	Date	Groundwater level below surface (ft)	Extent of trench below groundwater table (ft)
DT-1	.08	3.52/4.17	10/12/82	12.80	0.0
			11/9/82	8.10	0.0
			4/28/82	7.30	0.0
DT-2	.08	2.55	10/12/82	12.80	0.0
			11/09/82	8.10	0.0

1. Source: City of Tampa

TABLE 3-6. EQUIVALENT RAINFALL DEPTHS FOR DRAIN TRENCH CONTROL TESTS.

Test site	Area [1] (ac)	Runoff coefficient	Date	Calculated rainfall intensity (in/hr)	Test length (min.)	Equivalent rainfall depth (in.)
DT-1	1.08	0.38	10/12/82	0.20	27.5	0.09
			11/18/82	0.86	83.0	1.19
			4/28/83	0.29	240.0	1.16
DT-2	2.09	0.38	10/12/82	0.46	5.0	0.04
			11/16/82	0.20	73.0	0.25

1. Source: Task II.8.

Summary. The outflows observed at the end of the drain trenches indicate that a long-term hydraulic removal rate of 30 to 35 percent of the inflow is attained. This conclusion is based on the November 18, 1982 dry weather tests at both sites and the wet weather test on April 28, 1983 at site DT-1. Given the test lengths, it appears as though drain trenches can continue to achieve this removal rate during most rainfall events.

The dry or wet weather removal capacity of drain trenches is dependent on storm characteristics. A drain trench may perform better during a low-intensity storm in wet weather than a high-intensity storm in dry weather, as was shown by comparing the November and April tests at site DT-1.

The apparent poor performance of both drain trench sites during the October tests is more likely a function of short test lengths. Had the tests continued, a greater volumetric removal would have been obtained because the rate of outflow had not reached the rate of inflow. Even so, these tests indicate that drain trenches can not always be relied upon for large volumetric removals. Possible reasons for this include obstruction of the inlet pipe, hydraulic overloading from intense rainfall events, and saturated soil conditions.

It is of interest to note that the drain trenches performed consistently better than OBI's. The volume of storage in drain trenches is greater, but from Tables 3-1 and 3-4 it is clear that storage along the drain trench was utilized in most tests whereas storage around the OBI's was not. The primary reason for this may be the different design of the inlet chamber for these stormwater controls. The perforated pipe leading to the drain trench is located about 9 inches above the chamber bottom. If debris did collect in the inlet box, the elevation of the drain trench pipe would allow it to continue functioning and transporting water down the drain trench.

The OBI operation is dependent upon a clear surface above the gravel pack within the concrete box. If this is clogged, little water can get through to the storage area and, subsequently, the soil around it. The data collected on both OBI's and drain trenches demonstrates the importance of routine maintenance and clean-out to insure correct functioning of these controls.

Ditch Systems

Description and Summary of Tests. Ditch systems, or swales are surface stormwater runoff controls that act as conveyors of runoff to the storm sewer and provide for infiltration of runoff to reduce flows and loads to the storm sewer. Ditch systems are usually trapezoidal in shape with a depth varying from 1.5 to 2 feet and a surface width of 6 to 7 feet. The inverts of regular ditches are normally level with the inverts of culverts that connect the ditches. Undercut ditches have an invert that is up to 0.5 feet below the invert of culverts. This increases the storage area and drainage surface of the ditch.

Four ditch sites were selected for control testing in the northern portion of the City of Tampa and one site was selected in the southern portion (Figure 3-1). Soils found in four of the five test sites have good internal drainage characteristics. Detailed descriptions of the sites and the tests undertaken are presented in the Task II.8 report (2).

One to three tests were performed at all the ditch sites. The antecedent weather conditions were dry for all test dates in 1982. Test results are summarized in Table 3-7. For some tests a variable inflow rate was used. These are presented in the Table. The duration for which these variable flow rates occur is also presented. An additional test was requested by M&E in 1983 and performed by the City of Tampa at site DS-2 on May 5, 1983. This test followed a relatively wet 3-1/2 week period with an average daily rainfall of 0.10 inches. An inflow-outflow hydrograph for this additional test is presented in Figure 3-5.

Analysis of Results. A preliminary review of the results presented in Table 3-7 indicates that ditch systems can remove a significant amount of stormwater runoff. All the tests performed in late 1982 are from one to two hours duration. The additional test on May 5, 1983 at site DS-2 was carried out over a 3-1/2 hour period to observe long-term removal capacity. A rough comparison between the December 9 and May 5 test at this site can be made since their inflow rates are similar. The differences between the tests which should be noted are:

TABLE 3-7. SUMMARY OF CONTROL TESTS PERFORMED ON DITCH SYSTEMS(1)

Test site	Date	Inflow(2) (cfm)	Time from start of test when outflow begins (min)	Flow rate at end of test: Outflow (cfm)	Flow rate at end of test: Removal (cfm)	Flow rate at end of test: Removal (% of inflow)	Length of test(2) (min)
DS-1	11/24/82	3.4/9.1	43	5.7	3.4	37	25.0/62.5
	12/8/82	4.5/8.8 (3)	52	3.8	5.0	57	20.0/76.0
DS-2	11/24/82	4.3/15.2/2.5	48	4.7/0.0	10.5/2.5	69	45.0/65.0/10.0
	12/9/82	8.7 (3)	33	2.7	6.0	69	95.0
	5/5/83	8.2	28	4.7	3.5	43	210.0
DS-3	11/19/82	3.9/6.4	27	4.7	1.7	27	19.0/43.0
	12/8/82	8.1 (3)	22	4.3	3.8	47	64.0
DS-4	11/19/82	3.8	19	2.7	1.1	29	84.0
	12/8/82	5.3 (3)	25	2.3	3.0	57	72.0
DS-5	12/9/82	9.4	28	4.3	5.1	54	53.0

1. Source: Task II.8.
2. When two or more values are represented they indicate variable inflow rate during the test. The length of the test indicates the duration of each inflow rate.
3. Undercut ditches were simulated in these tests by raising the inflow weir height about 0.2 to 0.5 feet.

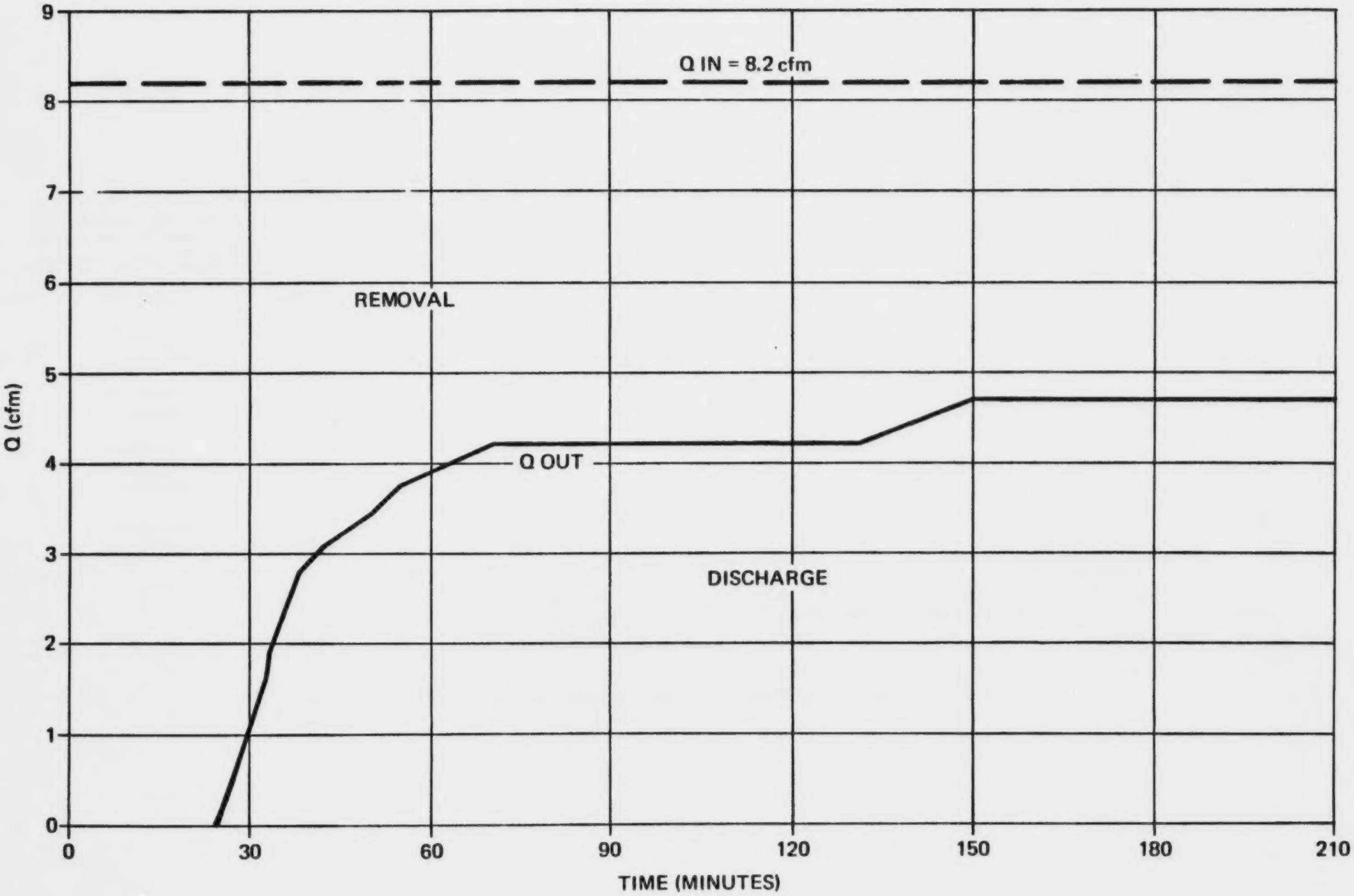


FIG. 3-5 INFLOW-OUTFLOW RESPONSE FOR MAY 5, 1983 CONTROL TEST AT SITE DS-2

1. The December 9 test was a dry weather test and the May 5 was a wet weather test;
2. The December 9 test simulated an undercut ditch resulting in a higher wetted perimeter along the ditch;
3. The December 9 test was shorter than the May 5 test.

These differences indicate that the December 9 test should produce better removal results due to the greater wetted perimeter and the drier antecedent conditions. These presumptions are correct. At the time of this test termination (95 minutes), the percent of inflow removed is 69 percent. From Figure 3-5, at 95 minutes, the May 5 removal rate was 4.0 cfm, or 48 percent of the inflow. At 210 minutes when this test was terminated, the removal rate had been reduced to 3.5 cfm, or 43 percent of the inflow. This small difference observed over 2 hours of testing indicates that test lengths of 1 to 2 hours on the ditch system sites produce reasonable estimates of short and long-term removal rates for dry weather conditions.

Groundwater level data for the ditch sites was not measured during the test dates. The only well measurements available were taken on November 9, 1982. These showed groundwater levels to be 3.18 feet below the surface for sites DS-1 through DS-3 and DS-5, and 6.34 feet below the surface for site DS-4. The monitor wells were at a distance of 0.7 to 1.36 miles from the ditch sites. Groundwater levels did not limit the removal capacity of these sites, and are not likely to because ditches are shallow and near normal ground elevation.

Soil Infiltration. Another method to compare the performance of the ditch systems during various tests is to calculate the variation of soil infiltration over the duration of each test. Table 3-8 presents the physical characteristics of the five ditch sites tested. The infiltration rate was calculated for three periods during the test. The first period reflects seepage to the soil before any outflow was measured, the second describes seepage during the period of fluctuating outflow and the third is a measure of infiltration at the end of the test when an apparent steady-state outflow was observed for a period of time. The surface storage in the ditch is accounted for in these calculations.

The calculated infiltration rates for the ditch tests are presented in Table 3-9. Some discrepancy is noted in the infiltration rates calculated for variable inflows such as on November 24 at site DS-1. In this test the inflow began as 3.4 cfm and was increased to 9.1 cfm before outflow was observed. An infiltration rate was calculated for both inflows as shown in Table 3-9. The assumption was made that infiltration was occurring along the entire length of the ditch, thereby yielding a low infiltration rate for the 3.4 cfm inflow and a higher one for the 9.1 cfm inflow. In reality, a low inflow of 3.4 cfm was probably absorbed quickly by the soil thus impeding water movement down the ditch. This limited the infiltration activity to a small portion of the ditch which would yield higher infiltration rates than those presented for this first period of

TABLE 3-8. PHYSICAL CHARACTERISTICS OF DITCH SYSTEMS

Site	Effective length (ft)	Slope (%)	Top width (ft)	Bottom width (ft)	Depth (ft)
DS-1	169	.46	6.0	1.0	1.5
DS-2	214	.42	6.0	1.0	2.0
DS-3	362	.75	7.0	1.5	1.5
DS-4	175	.47	7.0	1.5	1.7
DS-5	220	.43	6.0	1.5	1.5

TABLE 3-9. INFILTRATION RATES CALCULATED FOR DITCH SYSTEM CONTROL TESTS

		Infiltration rate (1) (in/hr)		
Test site	Date	Before outflow begins	Transition period	End of test
DS-1	11/24/82 (2)	14.5/38.6 (3)	26.4	10.9
	12/08/82	10.6/22.4	6.3	5.9
DS-2	11/24/82 (2)	9.4/25.1	18.1	16.1
	12/09/82	14.6	5.7	4.2
	05/5/83	17.9	6.3	5.5
DS-3	11/19/82 (2)	5.1/8.4	4.0	1.4
	12/8/82	7.8	2.2	2.3
DS-4	11/19/82 (2)	10.5	5.4	2.5
	12/8/82	9.5	3.2	3.3
DS-5	12/9/82	20.6	14.2	8.0

1. Assumes that infiltration occurs along entire effective length of ditch.
2. Undercut ditches simulated in these tests.
3. For variable inflow rates an infiltration rate was calculated for each flow rate.

the tests. Since details of the inflow progression along the ditch were not noted during the tests, it is not possible to take this factor into account.

The lowest infiltration rates were calculated at site DS-3. The likely reason for this is that the slope of the ditch at this site is 0.75 percent, much steeper than the other sites whose slopes were about 0.45 pecent. This supports the intuitive reasoning that flatter slopes improve soil drainage capabilities.

In general the calculated infiltration rates in Table 3-9 are higher than the 3.0 to 0.5 in/hr range of infiltration used to calibrate the SWMM model in Chapter 2. The SWMM model assumes that 0.25 inches of rain is lost to ponding in pervious areas. This loss occurs before the stormwater reaches the ditches which partially explains why higher infiltration rates were not necessary for model calibration. The major factor, however, as shown in model calibrations for the Norma Park basin, is that these ditches are capable of removing large amounts of runoff volume.

The performance of undercut ditches can be assessed from Tables 3-9 and 3-10. At site DS-1 the weir notch was raised 0.5 ft. above the ditch invert for the December 8 test. Although this test had lower infiltration rates overall, Table 3-10 shows that more volume was removed in the undercut simulation before outflow began. Also, the total volume removed during the test is 83 percent of the total inflow volume for the undercut simulation compared with 68 percent of total inflow for the normal ditch test.

TABLE 3-10. ADDITIONAL STATISTICS ON DITCH SYSTEM CONTROL TESTS

Test site	Date	Inflow (cfm)	Volume of inflow retained in ditch when outflow begins (ft^3)	Total volume of inflow over entire test period (ft^3)	Total volume of water removed during test (ft^3)
DS-1	11/24/82	3.4/9.1 (1)	248	551	443
	12/8/82	4.5/8.8	372	760	629
DS-2	11/24/82	4.3/15.2/2.5 (1)	240	1,204	956
	12/9/82	8.7	288	830	718
	5/5/83	8.2	230	1,724	974
DS-3	11/19/82	3.9/6.4	125	351	201
	12/8/82	8.1	178	510	381
DS-4	11/19/82	3.8 (1)	72	321	159
	12/8/82	5.3	133	384	303
DS-5	12/9/82	9.4	264	500	421

1. Undercut ditches simulated in this test.

Test site DS-4 was also simulated for regular and undercut ditches. Tables 3-9 and 3-10 show that a higher inflow rate was used for the undercut test, yet the infiltration rates were equal to or greater for the undercut simulation. The total volume removed was 79 percent of the total inflow volume versus 50 percent for the regular ditch test. Similar observations can be made for test site DS-3.

Equivalent Rainfall Depth. As with the drain trench tests, equivalent rainfall depths were calculated for the ditch system tests using the formula as described earlier. An approximate drainage area for each ditch system test site was obtained from the City of Tampa. All of the tests sites were in low-density residential areas; therefore a runoff coefficient of 0.38 was again applied. Table 3-11 presents the equivalent rainfall depths for these tests. The intensities in inches/hour vary from a high of 0.54 in/hr for test site DS-1 to a low of 0.10 in/hr at test site DS-3. The majority of intensities were between 0.12 and 0.20 in/hr which are very common for Tampa storms (3). Figure 3-6 presents a plot of measured hydraulic flow removal versus the calculated rainfall intensity for drain trench and ditch tests. It can be seen that the overall performance of undercut ditches is better than that of drain trenches. The performance of regular ditches, however, spans a wide range. Several additional points would be necessary to define a clear relationship between rainfall intensities and hydraulic removal for all types of controls.

TABLE 3-11. EQUIVALENT RAINFALL CALCULATIONS FOR DITCH SYSTEM CONTROL TESTS

Test site	Area [1] (ac)	Runoff coefficient	Date	Calculated rainfall intensity (in/hr)	Test length (min)	Equivalent rainfall depth (in)
DS-1	0.68	0.38	11/24/82	0.48	87.5	0.70
			12/8/82	0.51	96.0	0.82
DS-2	3.08	0.38	11/24/82	0.14	120.0	0.28
			12/9/82	0.12	95.0	0.19
			5/5/83	0.12	210.0	0.42
DS-3	1.59	0.38	11/19/82	0.16	62.0	0.16
			12/8/82	0.22	63.0	0.23
DS-4	0.82	0.38	11/19/82	0.20	84.0	0.28
			12/8/82	0.29	72.0	0.35
DS-5	0.77	0.38	12/9/82	0.54	53.0	0.48

1. Source: City of Tampa

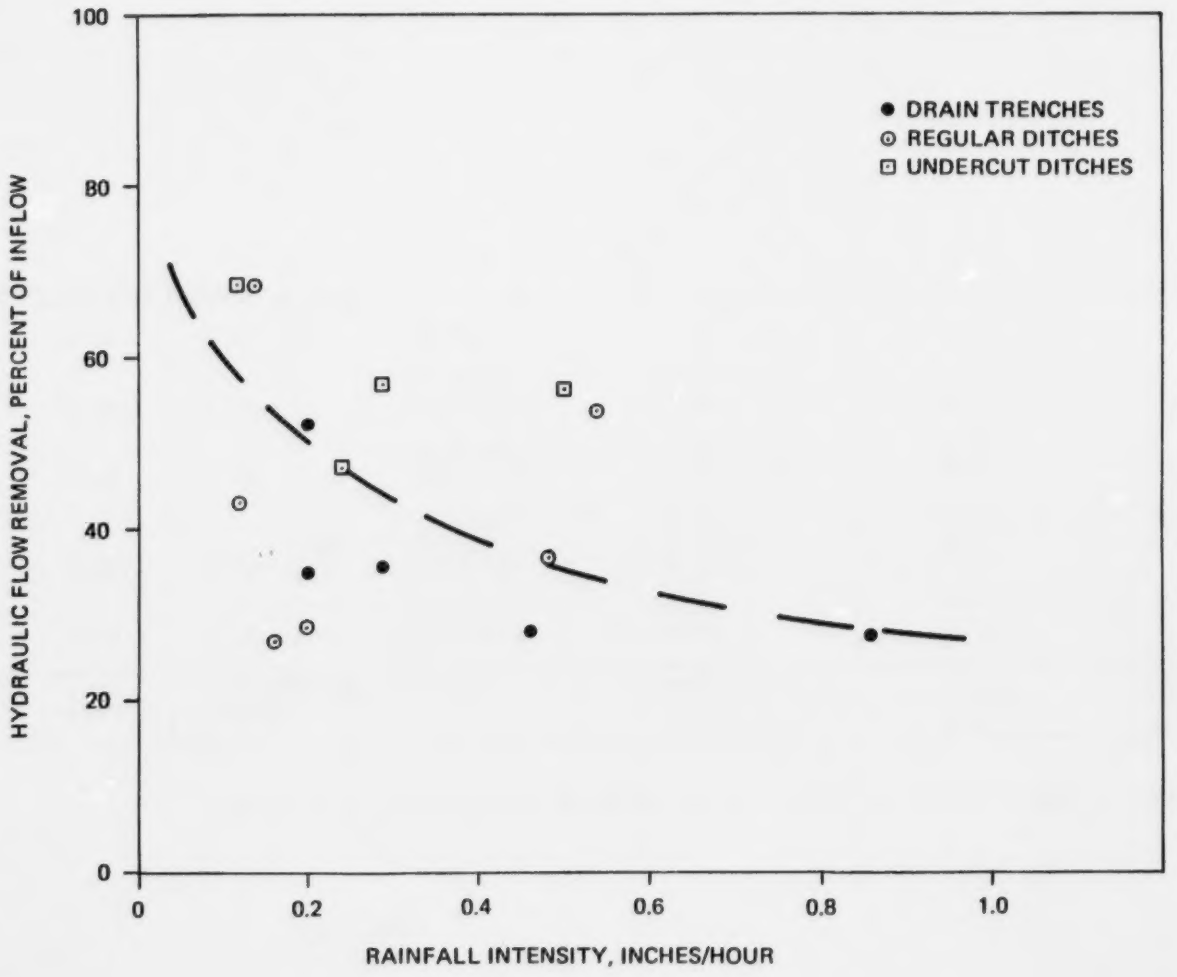


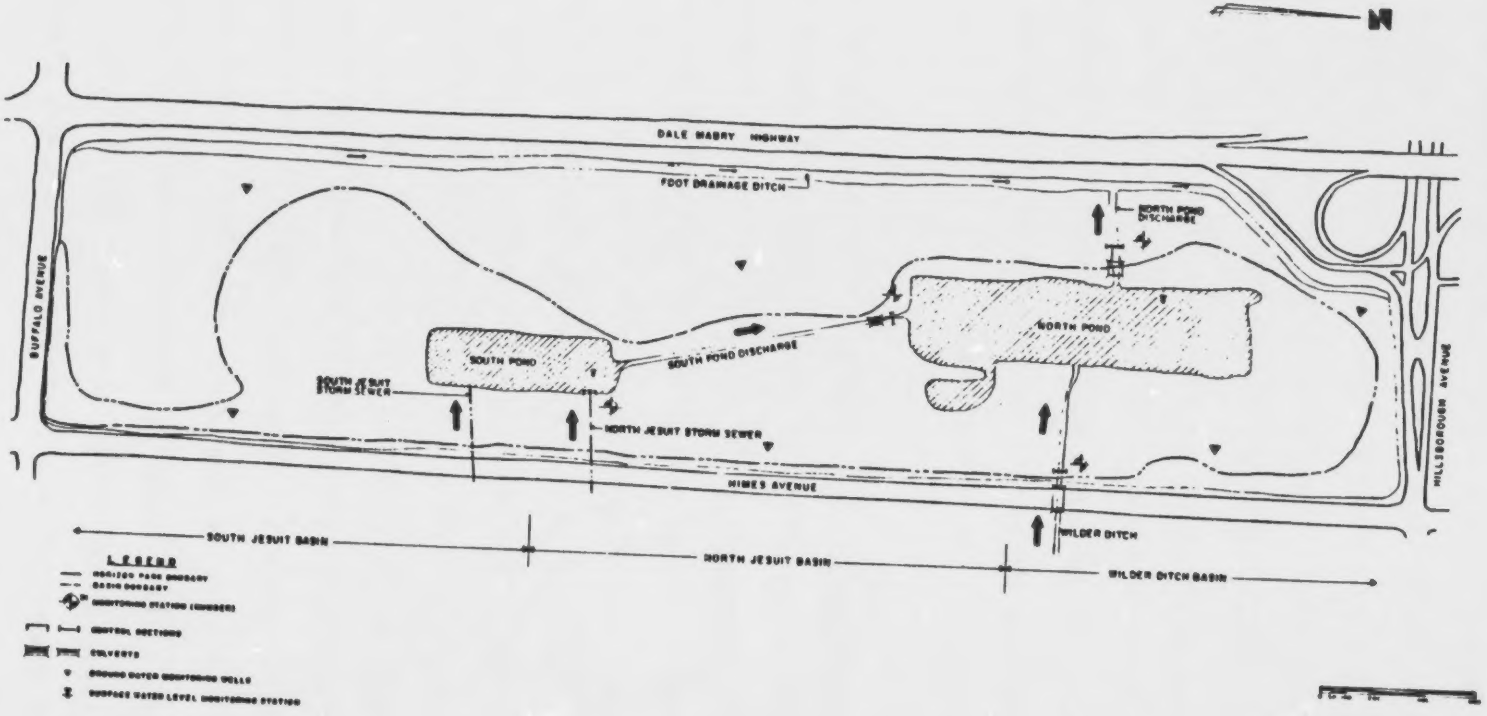
FIG. 3-6 PERCENT REMOVAL OF HYDRAULIC FLOWS FOR VARIOUS RAINFALL INTENSITIES

Summary. Based on the ditch system test results and the analysis presented above it appears that the ditch system removal capacity for dry and wet weather conditions is equal to or superior to that of drain trenches. From Table 3-7 the long term removal rate for regular ditches ranges from 27 to 69 percent of the inflow with an average of 43 percent. For undercut ditches the removal capacity is between 47 and 69 percent of the inflow with an average of 56 percent, an improvement over regular ditches. The ditches appear to be capable of sustaining these removal rates through most storm events.

Detention Ponds

Characteristics of Test Sites. A coupled system of detention ponds was selected for measurement and analysis of their capability to remove flow and pollutant loadings from stormwater runoff. The ponds, known as North Pond and South Pond, are located in Horizon Park and are shown in Figure 3-7. North and South Ponds have surface areas of about 9.1 and 2.9 acres, respectively. These areas are relatively large in comparison to other detention ponds in the Tampa area which are normally one acre or less in size (1).

North and South Ponds receive overland flow from the 126-acre Horizon Park Basin in which they are located. Storm sewer outfalls from the North and South Jesuit High School Basins lead into South Pond. The discharge from South Pond and stormwater runoff from Wilder Ditch Basin both enter North Pond. Additional details of the characteristics of these ponds are presented elsewhere (1,2).


N
DALE MABRY HIGHWAY
FOOT DRAINAGE DITCH
NORTH POND DISCHARGE
BUFFALO AVENUE
SOUTH POND
SOUTH JESUIT STORM SEWER
SOUTH POND DISCHARGE
NORTH POND
NORTH JESUIT STORM SEWER
HIMES AVENUE
HILLSBOROUGH AVENUE
WILDER DITCH
SOUTH JESUIT BASIN
NORTH JESUIT BASIN
WILDER DITCH BASIN
LEGEND
HORIZON PARK BOUNDARY
BASIN BOUNDARY
MONITORING STATION (NUMBER)
CONTROL SECTIONS
CULVERTS
GROUND WATER MONITORING WELLS
SURFACE WATER LEVEL MONITORING STATION

SOURCE: TASK II.8

FIG. 3-7 RETENTION/DETENTION POND SITE

Field Measurement Program. A field measurement program was developed to observe the flow regime of the two ponds and to characterize the pollutant concentrations of the various inflows and outflows of these ponds. A summary of the various data collected during the period of May 1982 to February 1983 is presented as follows:

1. Pond stage data - Stage data was collected by two automatic water level recorders located in the North and South Ponds.
2. Groundwater level data - Six monitoring wells were designed and located throughout Horizon Park. Weekly field readings were taken during the period of record.
3. Precipitation data - A precipitation gage was installed at the station by the ditch conveying South Pond discharge to North Pond. This gage automatically recorded precipitation during rainfall events.
4. Flow and water quality data - Four stations were selected to monitor inflows and outflows in the two ponds. Automatic flow meters and automatic samplers were installed at the following locations shown in Figure 3-7: a) the North Jesuit inflow to South Pond; b) the South Pond discharge to North Pond; c) the Wilder Ditch inflow to North Pond and d) the North Pond Discharge. Both composite and discrete samples were obtained for several rainfall events at these stations.

5. Laboratory treatability data - Some additional data were collected at the North Jesuit and Wilder Ditch inflows and in the North and South Ponds to study the time-varying settleability of pollutants in the ponds. A large sample of inflow to the ponds or pond water was collected during a runoff event. Water quality samples were taken from the surface of this large sample over a period of 21 to 168 hours and analyzed to observe the time-varying water quality of runoff or pond water.

Analysis of Rainfall/Runoff Events. As mentioned in the above summary, both composite and discrete samples of rainfall/runoff events were obtained at all four sampling stations during the period of May 1982 through February 1983. The number of events sampled at each station is presented in Table 3-12. Utilizing the common composite events or discrete events at these stations, a total of five events can be used for South Pond analysis and three events for North Pond analysis. A summary of these events is presented in Table 3-13.

The objective of this analysis is to determine the effect of the ponds on stormwater runoff quantity and quality. The hydraulic and water quality data will first be discussed for each event. The analysis of the effect of the ponds on hydraulics and water quality, and apparent pond removal rates for these events will then be summarized.

North and South Ponds - December 12, 1982 - A brief, intense storm occurred on this date, with 1.3 inches of rain

TABLE 3-12. SUMMARY OF COMPOSITE AND DISCRETELY SAMPLED RAINFALL/RUNOFF EVENTS AT NORTH AND SOUTH PONDS

Station location	No. of composite events	No. of discrete events(1)
North Jesuit Inflow	15	3
South Pond Discharge	0	5
Wilder Ditch Inflow	15	3
North Pond Discharge	0	4

1. A flow-weighted composite sample was calculated for these events.

falling in 1.3 hours. The inflow and outflow hydrographs measured at North Jesuit and Wilder Ditch inflows and the North and South Pond discharge are presented in Figure 3-8. This is the only event where discrete sampling was performed at both an inflow and outflow station for either pond. Figures 3-9 and 3-10 present the time-varying concentrations for several parameters at these stations. Figures 3-8 through 3-10 show significant flow and pollutant removal occurring for this event. Available storage at the start of this event was approximately 32,000 cu.ft. in South Pond and 143,000 cu.ft. in North Pond. The amount of hydraulic and pollutant removal that was observed in both ponds is increased by the filling of this storage volume.

South Pond - January 21,1983 - A 3-hour storm event during which 0.5 inches of rain fell resulted in the inflow and outflow hydrographs for South Pond as presented in Figure 3-11. A base

TABLE 3-13. CHARACTERISTICS OF NORTH AND SOUTH POND RAINFALL EVENTS

Date	Total rainfall (in.)	Storm duration (hrs.)	Pond levels at start of event (ft above MSL)		Available Storage (ft^3)		Discrete sampling location	Composite sampling location
			North pond (1)	South pond (2)	North pond	South pond		
12/12/83	1.3	1.3	23.14	24.60	143,000	32,000	North Jesuit South Pond North Pond	-
1/21/83	0.5	3.0	23.54	24.90	0	0	South Pond Wilder Ditch	North Jesuit
2/2/83	2.0	3.3	23.17	24.90	132,000	0	South Pond North Pond	North Jesuit Wilder Ditch
2/10/83	0.9	3.5	23.54	25.08	0	0	South Pond North Pond	North Jesuit Wilder Ditch
2/12/83	2.3	28.0	23.57	25.19	0	0	South Pond North Pond	North Jesuit Wilder Ditch

1. Outlet level at North pond is 23.5 ft. MSL.
2. Outlet level at South Pond is 24.8 ft. MSL.

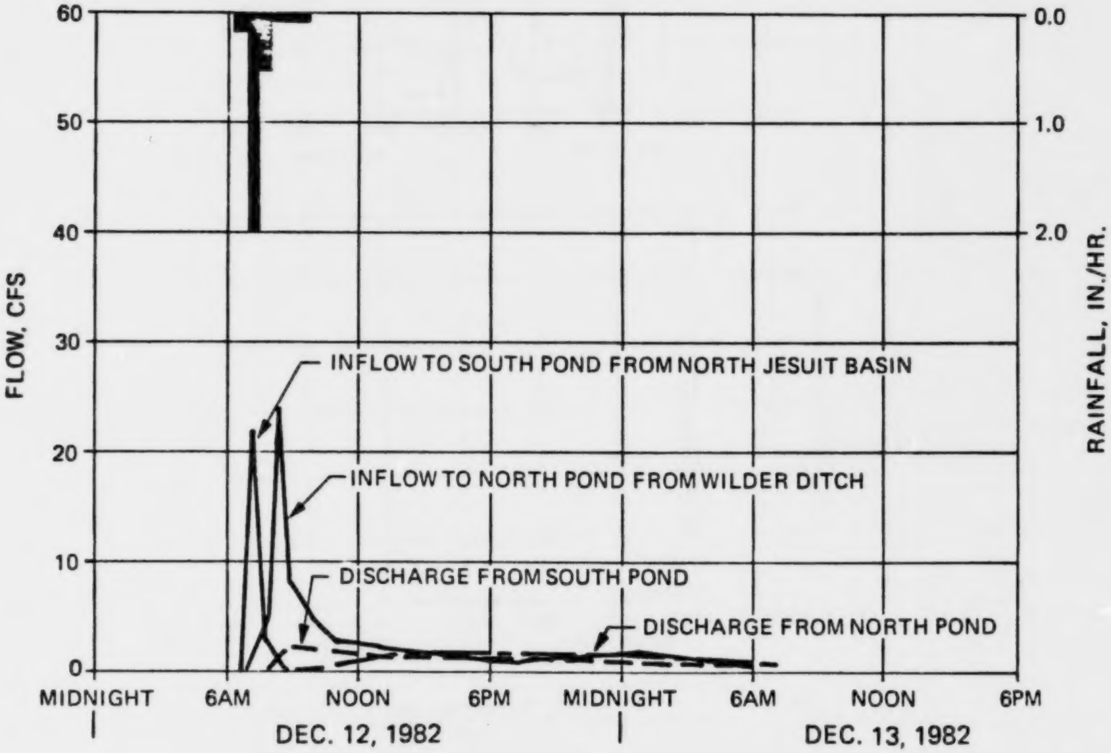


FIG. 3-8 INFLOW/OUTFLOW HYDROGRAPHS IN NORTH AND SOUTH PONDS - DECEMBER 12, 1982

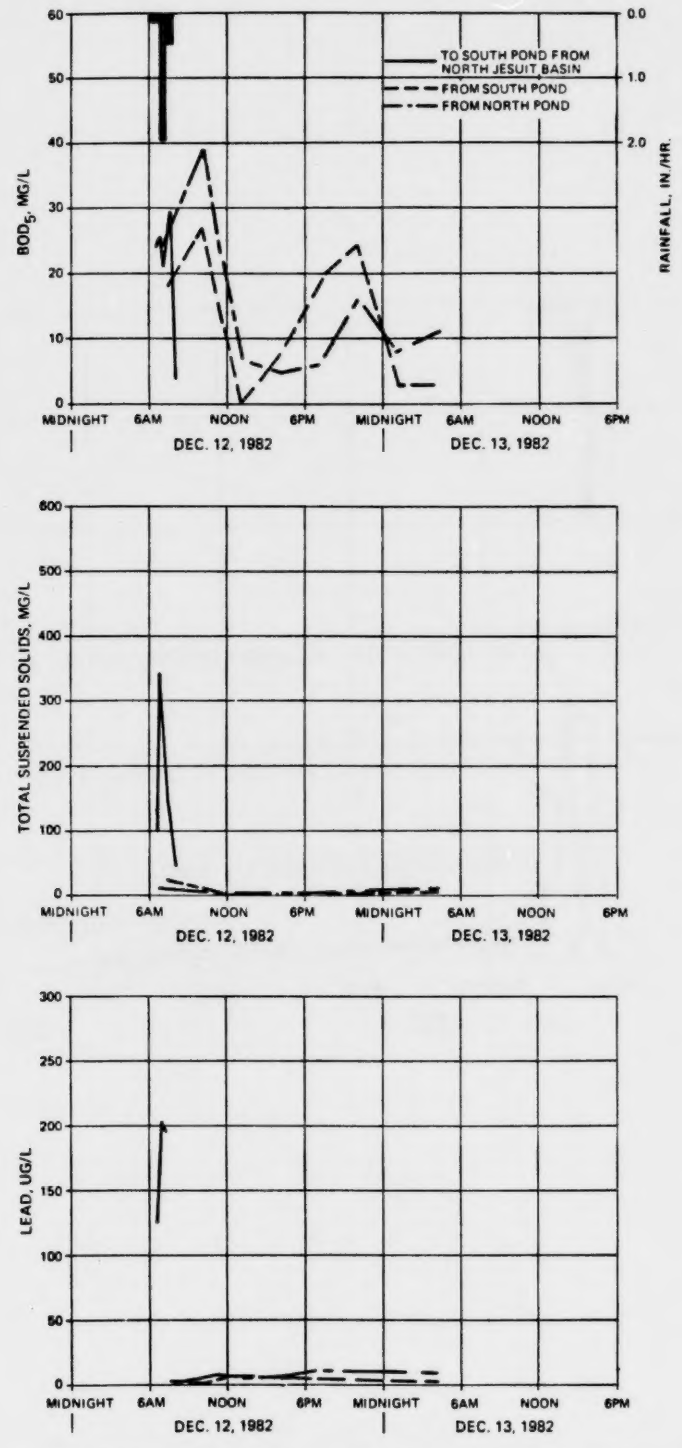


FIG. 3-9 INFLOW/OUTFLOW BOD$_5$, TSS AND LEAD CONCENTRATIONS IN NORTH AND SOUTH PONDS - DECEMBER 12, 1982

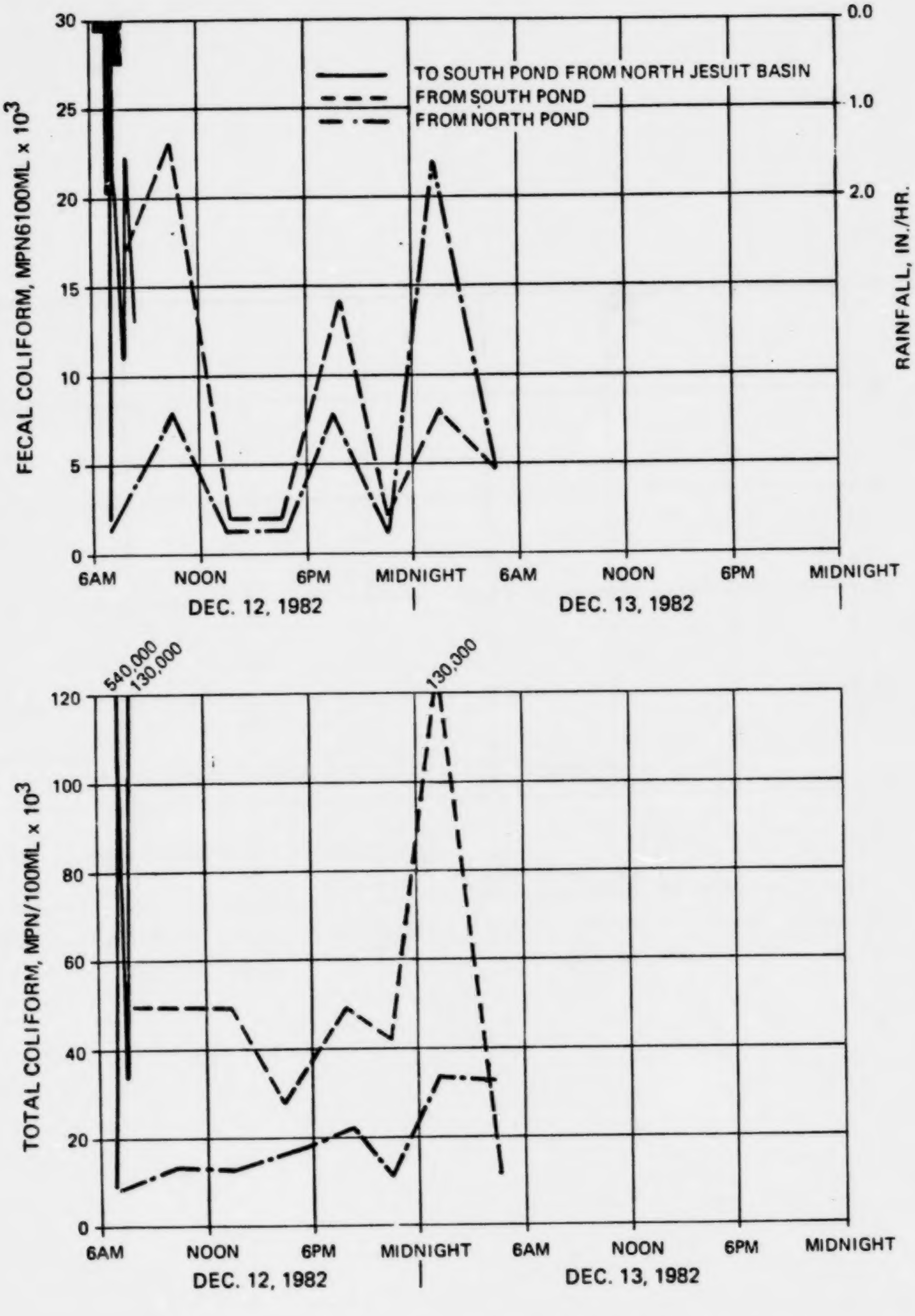


FIG. 3-10 INFLOW/OUTFLOW COLIFORM CONCENTRATIONS IN SOUTH POND - DECEMBER 12, 1982

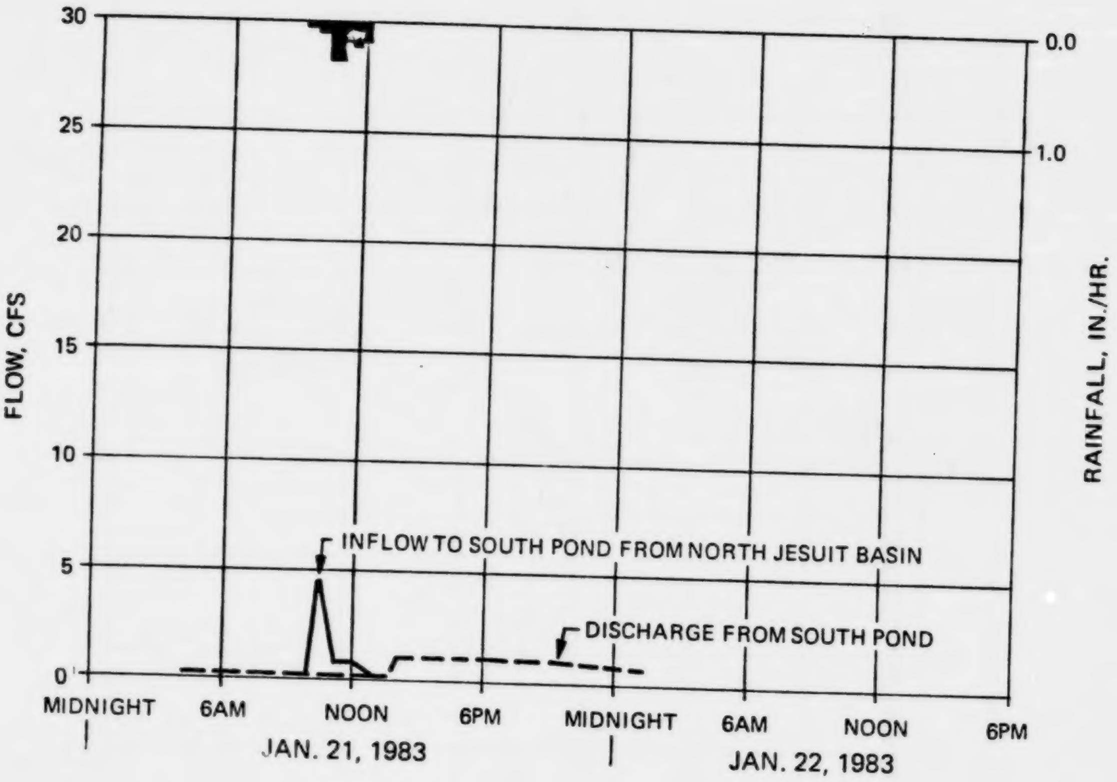


FIG. 3-11 INFLOW/OUTFLOW HYDROGRAPHS IN SOUTH POND - JANUARY 21, 1983

flow was present at the South Pond discharge; nevertheless, the peak inflow from North Jesuit was completely damped by the pond as shown by the outflow curve. It appears that the start time for the runoff from North Jesuit was erroneously recorded since the peak flow occurs prior to the peak of the storm. As mentioned in Chapter 2, the extreme local variability of rainfall in Tampa may also have caused this apparent discrepancy. No discrete water quality data were measured at the North Jesuit inflow station for this event.

North and South Ponds - February 2-3, 1983 - This was a short event of 3.3 hours with 2.0 inches of rainfall. Figure 3-12 presents the time-varying flows for all four stations. No storage was available in South Pond but the peak inflow rate from North Jesuit was significantly reduced. Approximately 130,000 cubic feet of storage was available in North Pond at the start of this storm. This storage would create a lag time between pond inflows and outflows. Based on a flow balance for this storm event, the North Pond should begin discharging around 8:00 AM, which is significantly earlier than the starting time given in Figure 3-12. In fact, the water quality plots of BOD_5 and total suspended solids presented in Figure 3-13 show that the automatic sampler at the North Pond outflow station was activated around 7:00 AM. Thus, the starting time is most likely in error. Since no discrete samples were taken at the inflow stations to the ponds, the water quality plots show only the pond discharges. These indicate that pollutant concentrations are about the same in each pond and follow similar trends.

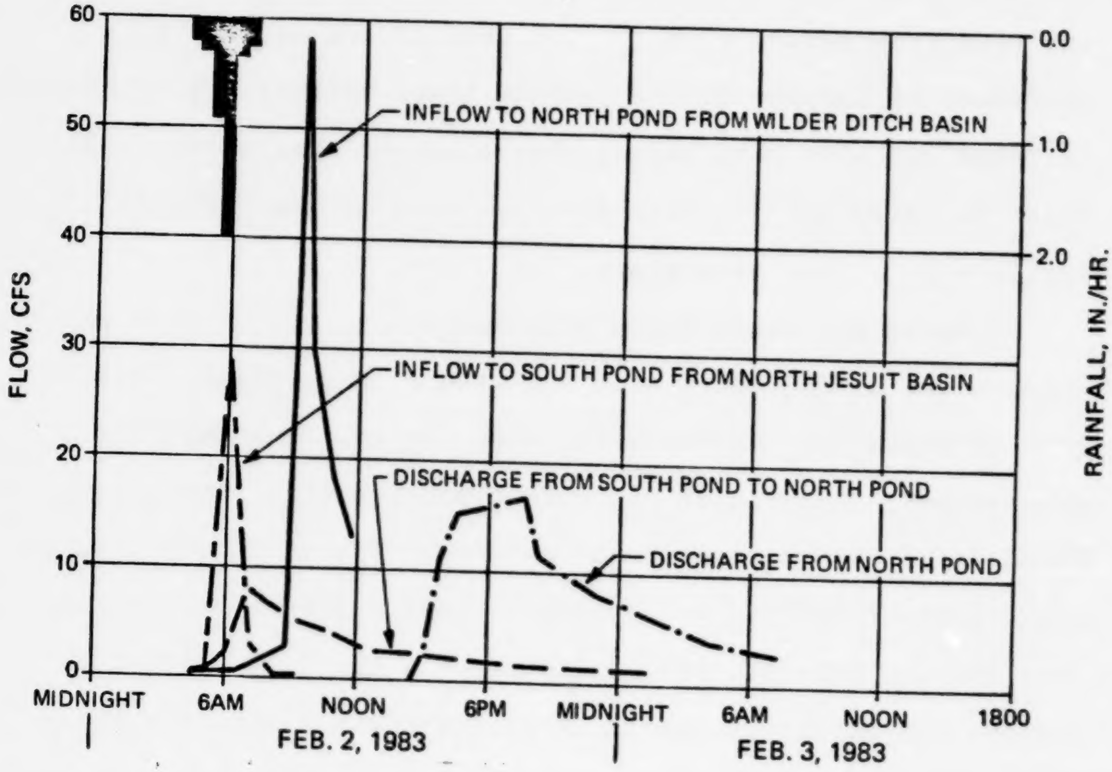
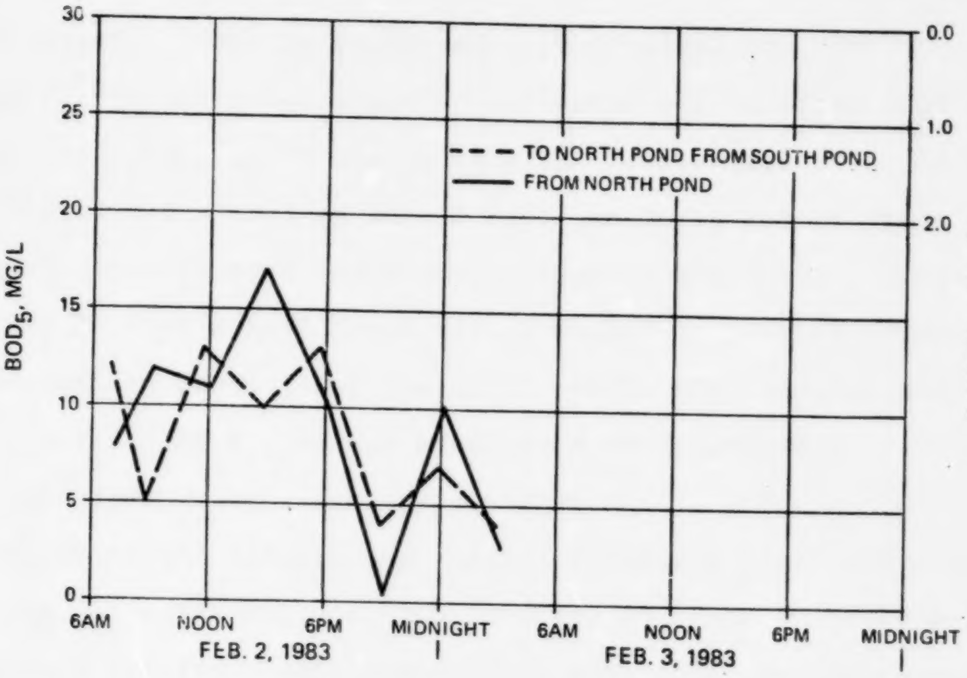


FIG. 3-12 INFLOW/OUTFLOW HYDROGRAPHS
IN NORTH AND SOUTH PONDS - FEBRUARY 2, 1983



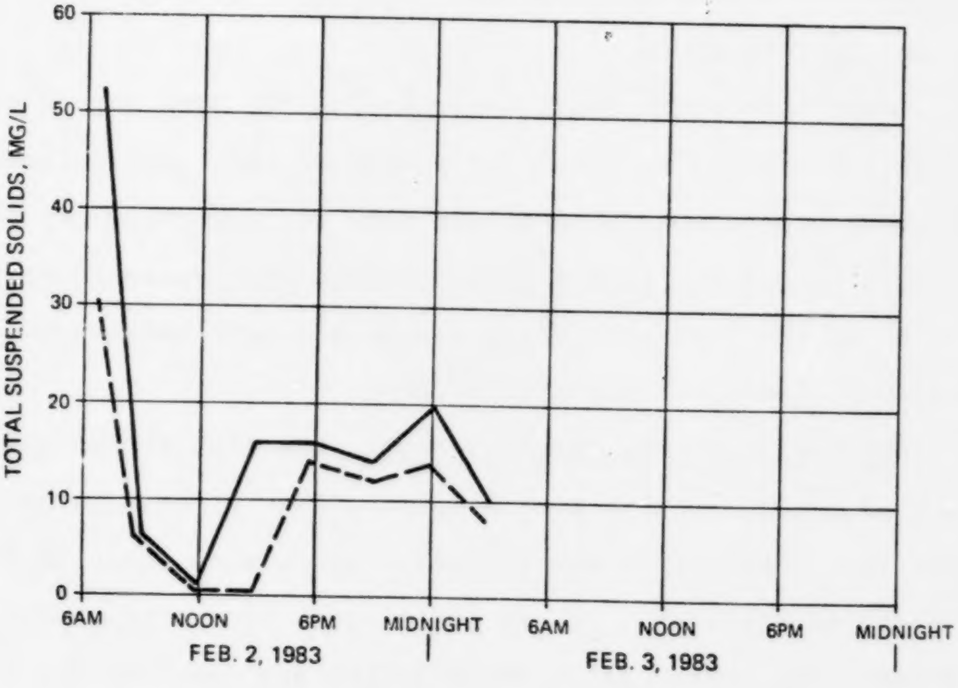


FIG. 3-13 OUTFLOW BOD$_5$ AND TSS CONCENTRATIONS FOR NORTH AND SOUTH PONDS - FEBRUARY 2, 1983

North and South Ponds - Februrary 10, 1983 - The flows at all four stations are presented in Figure 3-14 for this 3.5 hour, 0.9 inch rainfall event. Both North and South ponds were full at the start of the storm and both discharge soon after inflow begins. The outflow hydrograph for North Pond is questionable. The total volumes of inflow to the North Pond were 4.95 and 2.21 million gallons from Wilder Ditch and South Pond, respectively. The total discharge from North Pond was only 2.85 million gallons. A check of the groundwater levels shows that they were relatively high, therefore ruling out possible infiltration of the missing volume. No discrete data are available for the inflow stations. The pond outflow stations exhibited similar water quality trends as those seen in Figure 3-13 for the February 2, 1983 storm.

North and South Ponds, February 12-14, 1983 - This event lasted 28 hours during which 2.3 inches of rain were recorded. Both ponds were discharging a base flow of about 0.36 cfs at the start of the event. Figure 3-15 presents the hydraulic reponse measured at the four stations. Again, discrete samples were measured at the outflow stations only.

Analysis of Pond Effectiveness. The five storms described in the preceding section were sampled either discretely or as a composite. Total flows and pollutant loads were calculated at the measured stations. In the South Pond, two inflows were not measured - the South Jesuit basin inflow and the overland flow from the south Horizon Park basin. The flows and pollutant

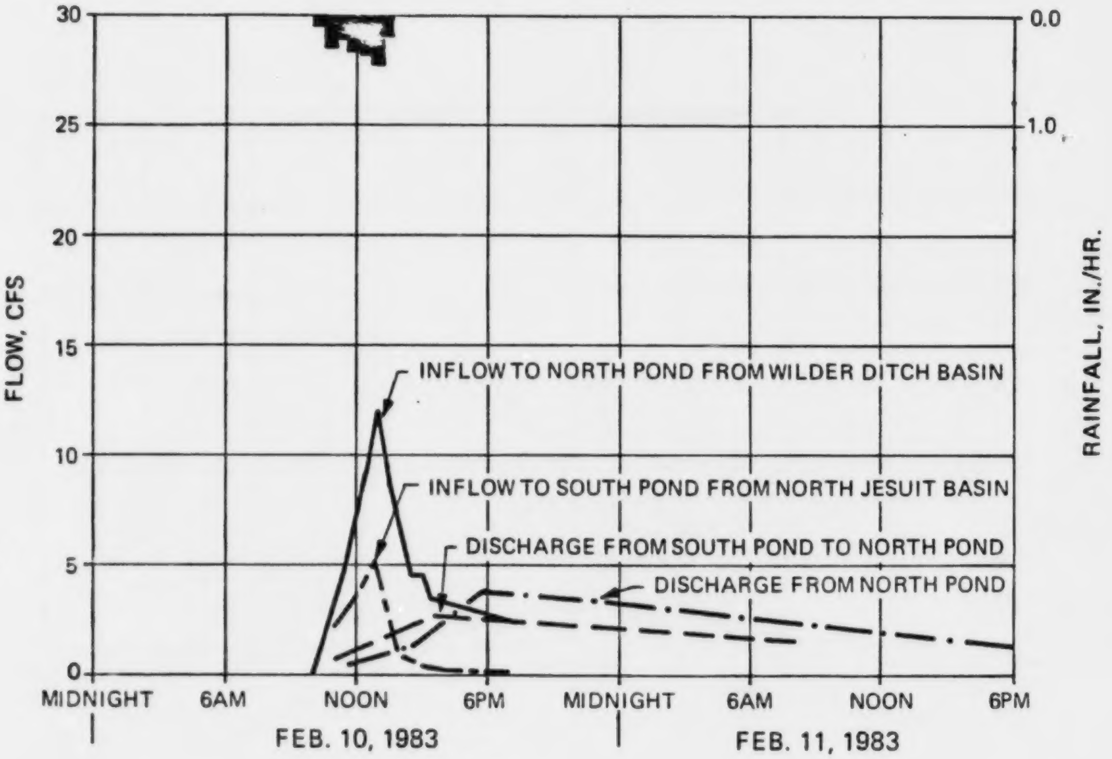


FIG. 3-14 INFLOW/OUTFLOW HYDROGRAPHS FOR NORTH AND SOUTH PONDS – FEBRUARY 10, 1983

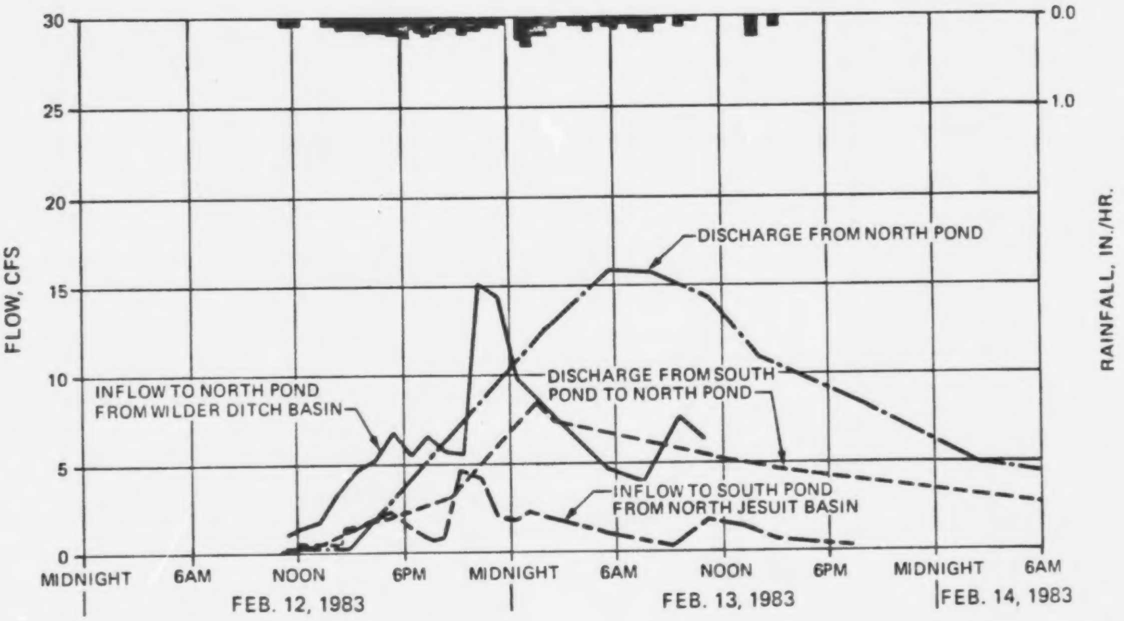


FIG. 3-15 INFLOW/OUTFLOW HYDROGRAPHS FOR NORTH AND SOUTH PONDS – FEBRUARY 12, 1983

loadings from these sources were estimated in order to assess the South Pond attenuation capacity. The flows and loads from the South Jesuit Basin were assumed to be 88 percent of the North Jesuit inflows. This value was based on comparison of total area and percent imperviousness of these two basins. The overland flow quantities were simulated using the SWMM model discussed in Chapter 2. Pollutant loads from the South Jesuit basin and overland flow were calculated by assuming pollutant concentrations to be the same as the mean concentrations from the North Jesuit basin.

The North Pond analysis also required an estimate of overland flows and loads. These were obtained using the SWMM model and assuming mean concentrations to be similar to Wilder Ditch basin concentrations. Direct loadings from the rain were computed using average concentrations measured at the Horizon Park rain gage. They were found to be insignificant for all parameters.

Table 3-14 summarizes the flows and loads and percent removals for the five storm events measured in South Pond. Table 3-15 presents the same information for two of the three storms sampled at North Pond. As discussed earlier, the inflows and outflows at North Pond for the February 10, 1983 storm did not balance, suggesting that the outflow volume from North Pond is erroneous.

The flow balance for both ponds as presented in Tables 3-14 and 3-15 indicate that the ponds are not very effective in

TABLE 3-14. POLLUTANT LOADS AND PERCENT REMOVAL IN SOUTH POND

Date	Total rainfall (in.)	Storm duration (hrs.)	Parameter	Inflow Loads (lbs) North Jesuit	Inflow Loads (lbs) South Jesuit(1)	Inflow Loads (lbs) Overland flow(2)	Inflow Loads (lbs) Rainfall	South Pond outflow loads (lbs)	Percent removal
12/12/83	1.3	1.3	Flow (mgal)	0.26	0.23	0.24	0.10(3)	1.09	-(4)
			BOD_5	50.0	44.0	46.0	-	100.0	29
			TSS	458.0	403.0	424.0	-	18.6	98
			T-Phosphate	2.0	1.8	1.9	-	0.9	84
			Total N	6.3	5.5	5.8	-	14.7	16
			Copper	0.02	0.02	0.02	-	0.01	83
			Lead	0.34	0.30	0.32	-	0.03	97
			Zinc	0.26	0.23	0.25	-	0.24	68
1/21/83	0.5	3.0	Flow (mgal)	0.06	0.05	0.05	0.04	0.31	-
			BOD_5	3.0	2.6	2.1	-	18.0	-
			TSS	<1.0	<1.0	<1.0	-	10.0	-
			T-Phosphate	0.14	0.12	0.10	-	0.2	44
			Total N	0.84	0.74	0.62	-	3.9	-
			Copper	0.003	0.003	0.002	-	0.005	38
			Lead	0.04	0.04	0.03	-	0.01	91
			Zinc	0.02	0.02	0.02	-	0.07	-
2/2/83	2.0	3.3	Flow (mgal)	0.54	0.48	0.27	0.16	3.94	-
			BOD_5	107.0	94.0	54.0	-	296.0	-
			TSS	395.0	348.0	198.0	-	328.0	65
			T-Phosphate	1.3	1.1	0.6	-	2.3	23
			Total N	6.2	5.5	3.1	-	28.3	-
			Copper	0.03	0.03	0.02	-	0.05	38
			Lead	0.57	0.50	0.29	-	0.17	88
			Zinc	0.45	0.40	0.22	-	1.02	5
2/10/83	0.9	3.5	Flow (mgal)	0.31	0.27	0.12	0.07	2.21	-
			BOD_5	67.0	59.0	26.0	-	129.0	15
			TSS	21.0	18.0	8.0	-	203.0	-
			T-Phosphate	0.3	0.26	0.1	-	1.1	-
			Total N	3.0	2.6	1.1	-	16.4	-
			Copper	0.009	0.008	0.004	-	0.10	-
			Lead	0.07	0.06	0.03	-	0.13	19
			Zinc	0.29	0.26	0.11	-	1.23	-
2/12/83	2.3	28.0	Flow (mgal)	1.20	1.06	0.28	0.18	2.17	20
			BOD_5	30.0	26.0	7.0	-	109.0	-
			TSS	<20.0	<20.0	<20.0	-	163.0	-
			T-Phosphate	0.9	0.8	0.2	-	0.9	53
			Total N	8.2	7.2	1.9	-	17.7	-
			Copper	0.04	0.04	0.009	-	0.03	66
			Lead	0.23	0.20	0.05	-	0.08	83
			Zinc	0.50	0.44	0.12	-	0.50	53

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1. Loads calculated as percentage of North Jesuit loads.
2. Loads calculated using SWMM and mean concentrations from North Jesuit Basin.
3. Insignificant pollutant loading.
4. Outflow load is higher than sum of all inflow loads.

TABLE 3-15. POLLUTANT LOADS AND PERCENT REMOVAL IN NORTH POND

Date	Total rainfall (in.)	Storm duration (hrs.)	Parameter	Inflow Loads (lbs)				North Pond outflow loads (lbs)	Percent removal
				Wilder Ditch	South Pond	Overland flow[1]	Rainfall		
2/2/83	2.0	3.3	Flow (mgal)	5.74	3.94	0.52	0.50	7.46	30
			BOD_5	476.0	296.0	48.0	-[2]	662.0	19
			TSS	2,668.0	328.0	22.0	-	933.0	69
			T-Phosphate	12.4	2.3	1.2	-	6.2	61
			Total N	78.0	28.3	2.1	-	79.0	27
			Copper	0.34	0.05	0.01	-	0.80	-[3]
			Lead	6.53	0.17	0.08	-	1.11	84
			Zinc	3.81	1.02	0.12	-	2.11	57
2/12/83	2.3	28.0	Flow (mgal)	7.27	2.17	0.33	0.57	11.7	-
			BOD_5	<121.0	<109.0	<5.0	-	585.0	-
			TSS	726.0	163.0	33.0	-	1,171.0	-
			T-Phosphate	4.2	0.9	0.2	-	6.8	-
			Total N	62.9	17.7	2.9	-	113.2	-
			Copper	0.13	0.03	0.006	-	0.25	-
			Lead	0.94	0.08	0.04	-	0.47	56
			Zinc	1.81	0.50	0.08	-	1.66	30

---

1. Loads calculated using SWMM and mean concentrations from Wilder Ditch Basin.
2. Insignificant pollutant loading.
3. Outflow load is higher than sum of all inflow loads.

removing hydraulic flows. Part of the reason for this is that storage was available for only one storm event in South Pond and two events in North Pond. Since pond levels were measured every day during the field measurement program, the frequency distribution of available storage was determined. Table 3-16 presents the storage volumes for North and South Pond obtained from daily pond levels recorded during the May 1982 through February 1983 field measurement program in terms of percent of time that they are exceeded. For example, storage volume is available only 45 percent of the time in North Pond and the available storage volume is over 100,000 cu. ft. about 23 percent of the time. Similarly, storage in South Pond is available less than 20 percent of the time and never exceeded 24,000 cu. ft. during the field measurement program.

With both ponds full more than 50 percent of the time, their main hydraulic function is to reduce and delay the peak inflow rate. The reduction of peak inflow was caclulated for both ponds from Figures 3-8, 3-11, 3-12, 3-14 and 3-15. For South Pond, the peak inflow rate from the South Jesuit basin was assumed to be 88 percent of the recorded peak inflow from the North Jesuit basin. SWMM model simulations provided a peak flow rate for overland flow to both ponds. The reduction of peak inflow for detention ponds ranges from 10 to 90 percent based on available data. The higher values correspond to events with available pond storage such as that of December 12, 1982, whereas the lower values reflect situations of full or overflowing ponds at the start of a rainfall event.

TABLE 3-16. FREQUENCY DISTRIBUTION OF OBSERVED STORAGE VOLUMES IN NORTH AND SOUTH PONDS

Percent of time exceeded	Storage volume (ft^3)	
	North pond	South pond
0	230,000	24,000
5	185,000	11,500
10	168,000	5,500
15	150,000	1,200
20	122,000	0
25	93,000	
30	63,000	
35	20,000	
40	2,000	
45	0	

The water quality results presented in these Tables reflect a wide range of removals for both ponds. The results indicate that pond performance is dependent on the intensity and duration of the storm events as well as the available storage volume. For example, at South Pond, the loads from the events of December 12, 1982 and February 2, 1983, both high-intensity, short-duration events, are significantly reduced. About 32,000 cu. ft. of storage was available on December 12, however no storage was available on February 2. The other three events were all weak intensity storms and, except for metals, few other pollution loads decrease as they pass through the pond. North Pond exhibits the same performance, with pollutant removals of 20 to 80 percent for the high-intensity storm event of February 2, 1983, for which about 130,000 cu. ft. of storage were available. Only a few metals decreased in the long, low-intensity event of February 12-14, 1983.

Table 3-17 presents the range of removals for each of the parameters listed in Tables 3-14 and 3-15. The highest observed removals were for lead and total suspended solids with copper, zinc, and total phosphate following. This indicates that the stormwater loads associated with particulate matter, such as solids and metals will tend to settle upon entering a detention pond. BOD_5 and total nitrogen exhbited the lowest range of removals, suggesting that either they exist in soluble form and do not settle, or in-pond chemical reactions tend to increase their concentrations. Lead attained a consistently high removal rate during most of the events.

Treatability Studies. To further observe the removal capability of detention ponds, some additional data were obtained at the North Jesuit and Wilder Ditch inflows and in the North and South Ponds. About 120 liters of stormwater runoff or pond water was collected at one time during a storm. This volume was kept in a container for several days during which samples were taken from the surface and analyzed for pollutant concentrations. Such tests were carried out twice for the North Jesuit and Wilder Ditch inflows and once for pond water composited from several locations in North and South Ponds. The dates of the tests and the characteristics of the rainfall events associated with them are presented in Table 3-18. The stations sampled are also given for each test. The in-pond test dates were different from the inflow test dates, thus direct comparison of time-varying concentrations in runoff and pond water from the same event cannot be made.

TABLE 3-17. DETENTION POND REMOVAL CAPACITIES

Parameter	Range of observed removal (%)	Average removal (%)
Flow(1)	23 - 96	69
BOD_5	0 - 29	9
TSS	0 - 98	33
Total Phosphate	0 - 84	35
Total Nitrogen	0 - 27	6
Copper	0 - 83	32
Lead	19 - 97	74
Zinc	0 - 68	30

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1. The given percentages refer to a reduction of peak inflow rather than actual removal of flows.

TABLE 3-18. STORM EVENT DATA FOR TREATABILITY STUDY SAMPLES

Date	Total rainfall (in.)	Storm duration (hrs.)	Antecedent dry period(1) (days)	Locations sampled
12/12/83	1.25	1.33	39	North Pond South Pond
1/20/83	0.70	5.5	38	Wilder Ditch North Jesuit
2/2/83	2.00	3.25	12	Wilder Ditch North Jesuit

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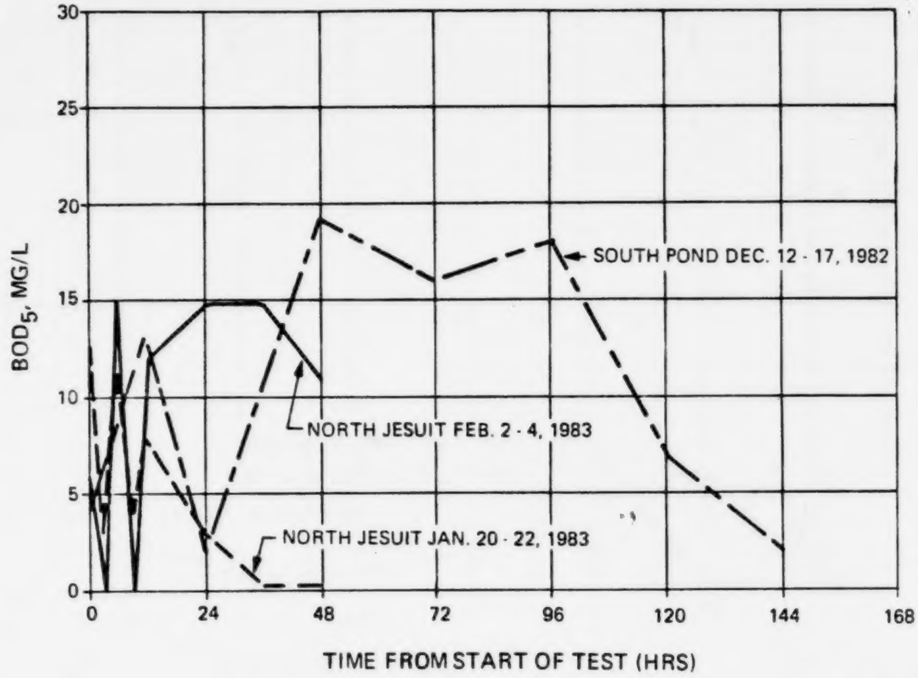
1. Days since last storm with 0.2 inches or more rainfall.

Figure 3-16 presents time-varying BOD_5 concentrations in North and South Pond and the stormwater inflow samples. The BOD_5 concentrations increase in the Wilder Ditch sample of February 2, 1983. The same is true in the North Jesuit sample. These results may be due to organic activity in the runoff sample. Over a period of six days both in-pond samples exhibit a general decrease of BOD_5 for the December 12-17 period and the samples at Wilder Ditch and North Jesuit collected January 20 exhibit similar results. The hydraulic retention time of these ponds for typical storm events are on the order of 1.0 to 1.5 days; thus BOD_5 removals are not expected to be high. This is verified in the data in Tables 3-14 and 3-15.

Figure 3-17 presents TSS concentrations for the two ponds and their inflows . Overall, the concentrations are very low, both in the pond samples and the stormwater runoff samples for all three events. Significant settling is not observed, although some of the initial concentrations are slightly higher.

The total nitrogen concentrations in Figure 3-18 indicate that there is relatively little change in in-pond concentrations over a long period of time after storm events.

The time varying concentrations of lead are presented in Figure 3-19. This Figure indicates a clear pattern of lead removal by the pond. North and South Pond concentrations for the December 12 storm are very low, indicating that lead settles out in the pond. Figure 3-20 for zinc concentrations also exhibits some reduction in concentration, but not as dramatic as lead.



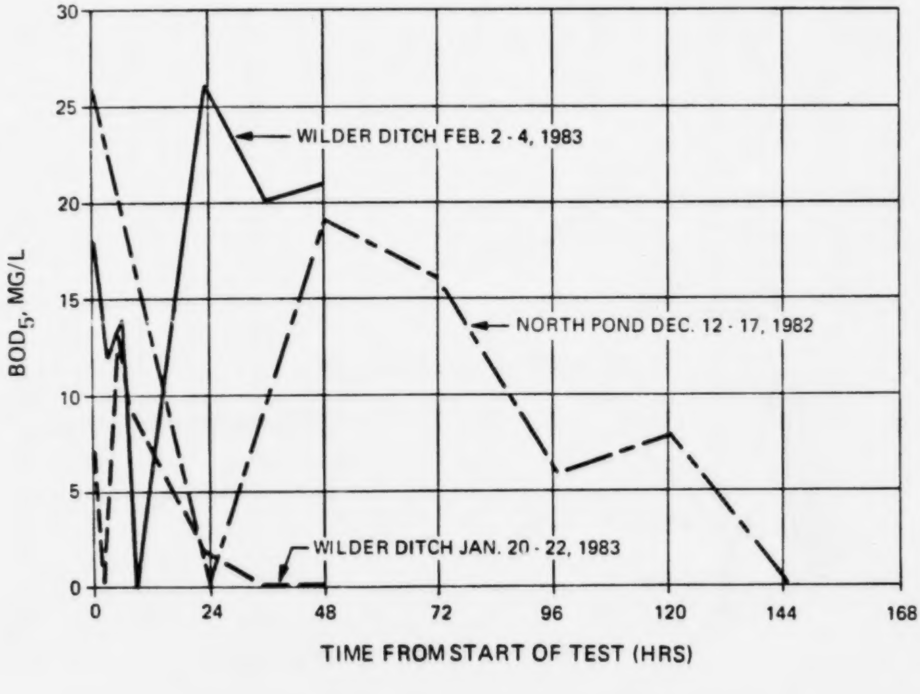


FIG. 3-16 TIME-VARYING CONCENTRATIONS OF BOD_5 IN STORMWATER RUNOFF AND POND IN-SITU SAMPLES

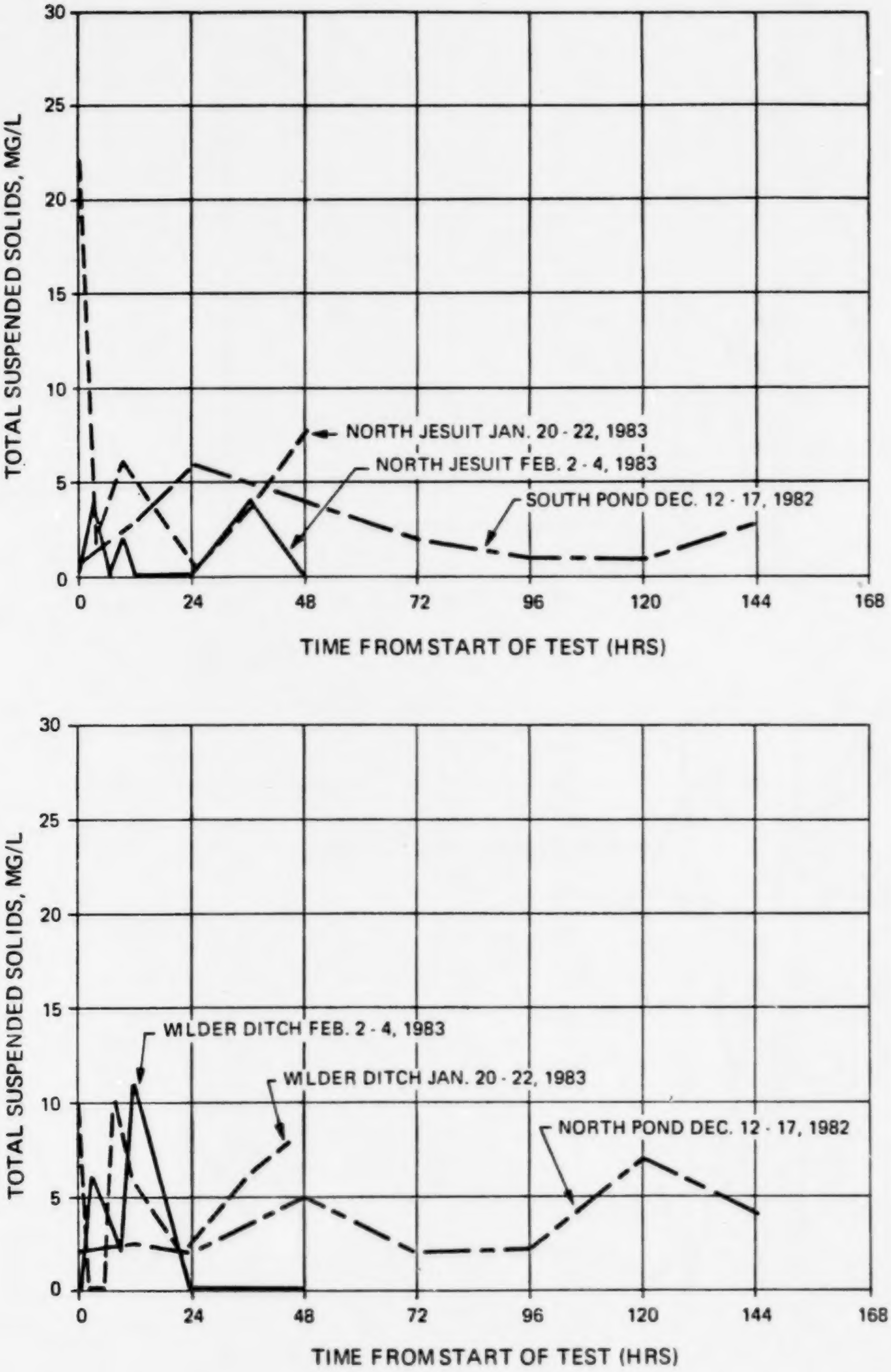


FIG. 3-17 TIME-VARYING CONCENTRATIONS OF TSS
IN STORMWATER RUNOFF AND POND IN-SITU SAMPLES

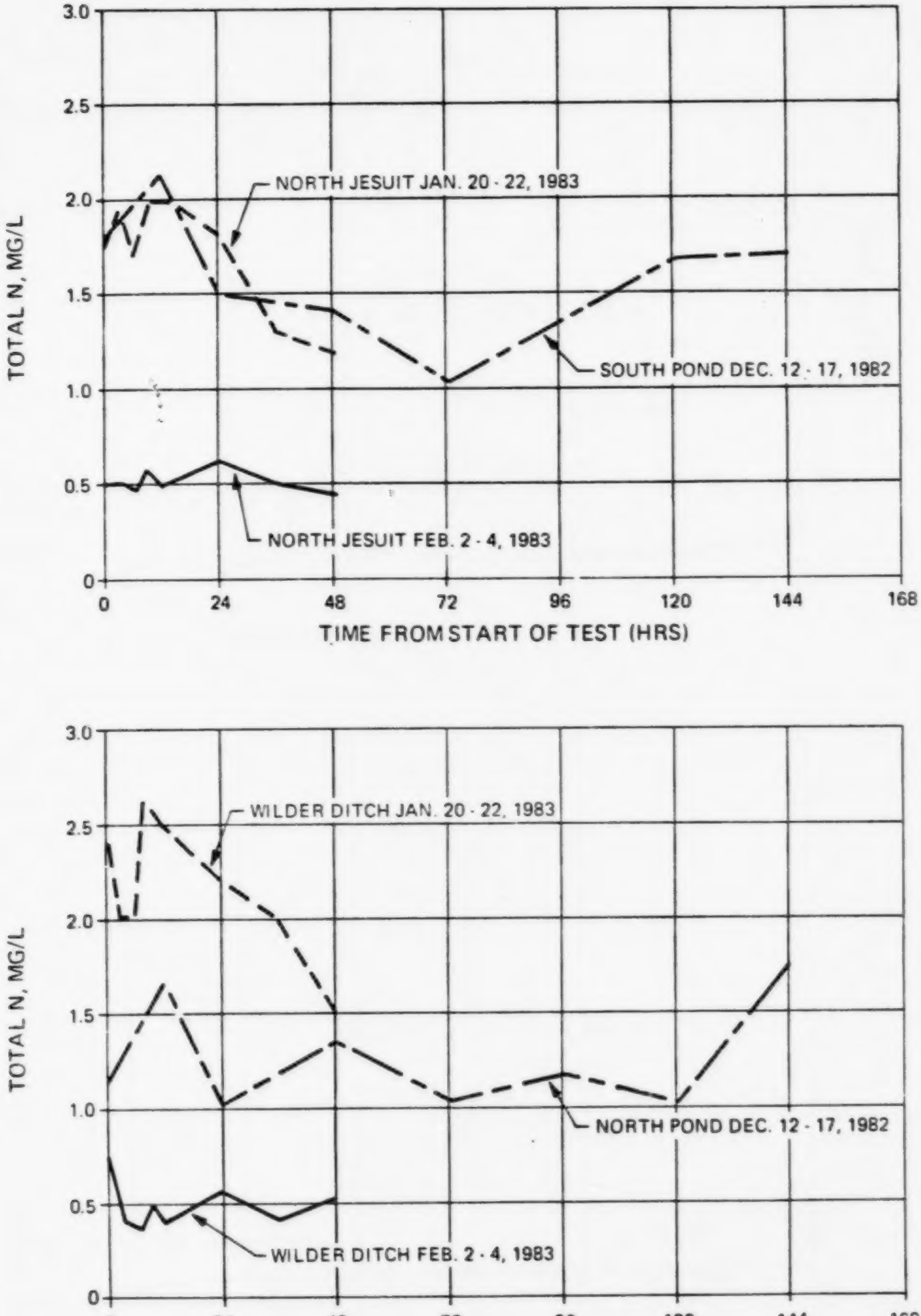
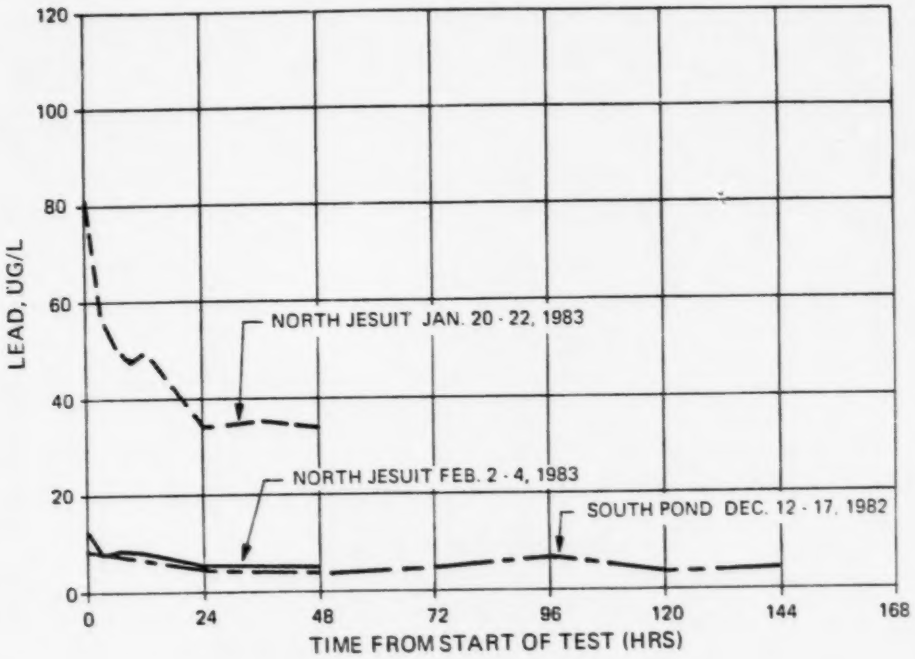


FIG. 3-18 TIME-VARYING TOTAL NITROGEN CONCENTRATIONS IN STORMWATER RUNOFF AND POND IN-SITU SAMPLES



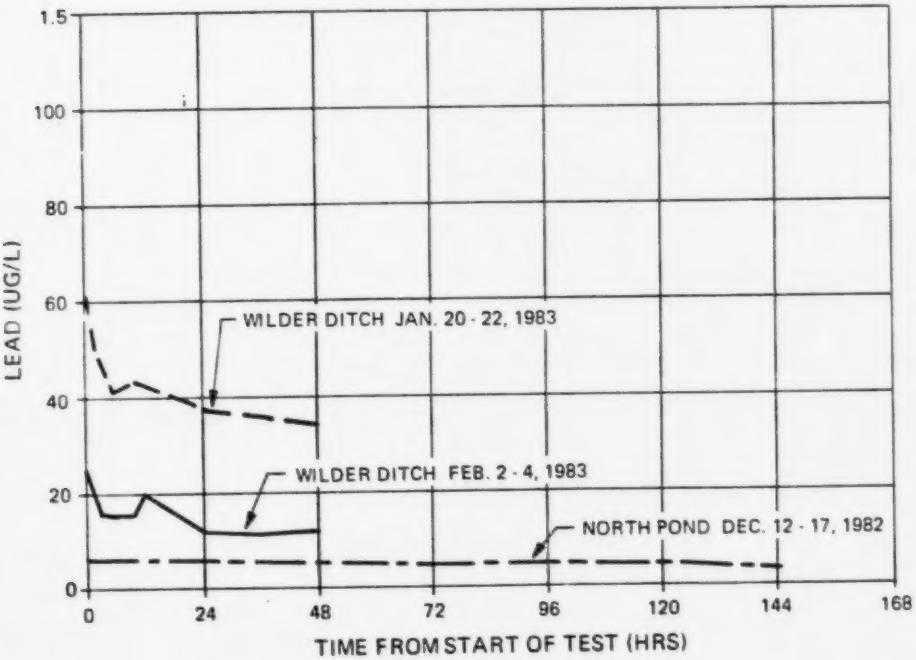


FIG. 3-19 TIME-VARYING LEAD CONCENTRATIONS IN STORMWATER RUNOFF AND POND IN-SITU SAMPLES

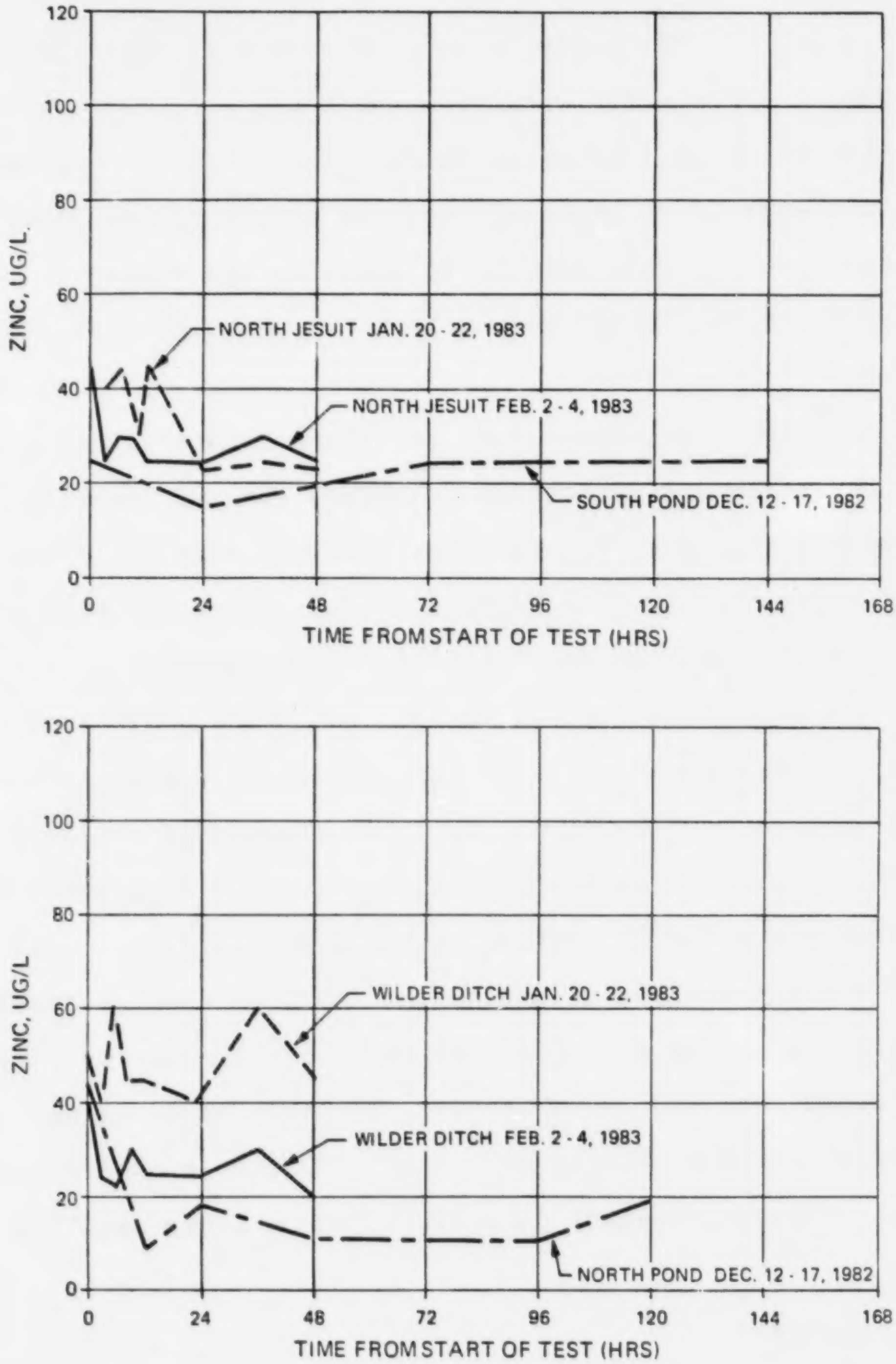


FIG. 3-20 TIME-VARYING ZINC CONCENTRATIONS IN STORMWATER RUNOFF AND POND IN-SITU SAMPLES

Summary. The analysis of available data from the field measurement program for detention ponds has produced some insight into the performance of these stormwater controls. The detention ponds are capable of reducing the peak runoff volume from stormwater runoff particularly if dead storage volume is available. Such storage volume is not frequently observed, however.

Available storage volume and storm intensity and duration characteristics all affect the pollutant removal capability of the detention ponds. It was noted that the best pollutant removals occurred for two high-intensity, short-duration storms for which some storage was available in the ponds.

Pollutant removal was highest for those pollutants associated with particulate matter and lowest for pollutants that are present in soluble form. Lead was removed effectively during the storm events. The treatability test results verified that lead from storm runoff settles, usually mostly within 1 to 2 days. BOD was exerted in the tests, although much of the BOD which enters during a storm event would be expected to escape. Other parameters did not show clear patterns of settling. In-pond water samples usually reflected a "steady-state" concentration of most pollutant parameters over a 6-day sampling period.

Cost Analysis

In previous sections the results of tests performed on various stormwater controls were presented and analyzed to obtain an estimate of their hydraulic and pollutant attenuation and removal characteristics. In this section the capital and O&M costs, provided by the City of Tampa, are presented for each stormwater control. The cost estimates were based on the City's own budget calculations and production estimates and the Jorgensen Work Management System (4). This is a budgeting method that provides guidance for breakdown and cost quantification of tasks. The costs presented are compared to the removal benefits of each control.

Table 3-19 presents a summary of long-term removal rates for the four stormwater controls. Since pollutant quality was only measured in the detention pond studies, the assumption is made that pollutant removal will be equal to hydraulic removal for open bottom inlets, drain trenches, and ditches. It should be noted that the hydraulic attenuation rate for detention ponds reflects a reduction in the peak discharge compared to the peak inflow rate. Based on pond level measurements, available storage in the ponds is limited. Therefore, it appears that the primary hydraulic function of detention ponds is to reduce the peak flow rate to the storm sewer.

The available capital and O&M costs for the various stormwater controls are presented in Table 3-20. They are described briefly in the following paragraphs.

Open Bottom Inlets. OBI costs are based on the unit costs for a standard inlet, clean river gravel and filter fabric. The construction cost of an open bottom inlet, estimated at $1,175,

TABLE 3-19. SUMMARY OF LONG-TERM REMOVAL RATES FOR VARIOUS STORMWATER CONTROLS

Stormwater control	Long-term hydraulic removal rate (% of inflow rate)[1]	Long-term pollutant removal rate (% of inflow load)
Open bottom inlets	0 - 5	0 - 5
Drain trenches	30 - 35	30 - 35
Ditch systems - regular	30 - 70	30 - 70
Ditch systems - undercut	50 - 70	50 - 70
Retention/Detention ponds	20 - 95	0 - 30 (soluble pollutants) 0 - 90 (particulate matter)

1. The hydraulic removal rate for detention ponds is the percent reduction or attenuation of the peak inflow rate.

TABLE 3-20. SUMMARY OF STORMWATER CONTROL COSTS[1]

Stormwater control	Capital unit costs ($)	Maintenance costs per year ($)
Open bottom inlets	1,175/OBI	34/OBI
Drain trenches	21.13/lin. ft.[2]	26.70/drain trench
Ditch systems/swales	5.69/lin. ft.	.06 - .10/lin. ft.
Retention/detention ponds	4.50/cu.yd. (excavation) 4.60/lin. ft. (fencing) 2.70/sq. yd. (sodding)	136/acre (mowing)

1. City of Tampa Fiscal Year 1983 estimates unless otherwise noted.
2. 1982 unit cost.

is only $125 more than that of a standard inlet. This difference reflects the additional cost of the gravel jacket surrounding the concrete inlet. A biannual OBI maintenance program was assumed which would include vacuuming the inlet and replacing the gravel. The annual maintenance cost would be approximately $34 per open bottom inlet.

Drain Trenches. A uniform trench cross-section 3.0 ft. wide, 2.0 ft. deep with an 8-inch diameter perforated pipe is used by the City to simplify the design and construction process. A 1982 unit cost of $21.13/lin. ft. has been provided for this type of control. Maintenance of drain trenches includes cleaning the culvert by jet and vacuuming the inlet about twice a year. For a 75 ft. trench extending from an inlet the annual maintenance cost would be $26.70. A maximum usage of two 75 ft. trenches extending from one inlet would cost $42.40 to maintain.

Ditch Systems. Yearly equipment and labor costs were divided by an assumption of yearly production of ditch lengths to obtain an excavation cost per lineal foot. Surveying and sodding costs were added to the excavation costs, yielding a unit cost of $5.69/ft. of regular or undercut ditches. Estimates of maintenance costs for minor regrading at intervals of 5 years are about $.52 per foot.

Retention/Detention Ponds. Retention pond costs are the most variable of all the storwater controls due to the variety of size, price and condition of existing land, and the range of auxiliary structures necessary for the proper utilization of the site. The City of Tampa provided costs independent of specific projects, namely excavation, sodding and fencing costs. It is not possible to develop an area/cost relationship due to the variability of pond depths (dependent on the groundwater table, shape and area available for storage).

The annual cost of mowing a detention pond is given to be $136 per acre. The costs of fence and structure maintenance, washout repairs and other operations are not available.

Comparison of Stormwater Control Costs and Benefits. A concise cost/benefit analysis can be made for three of the four types of stormwater controls. Open bottom inlets, though not very expensive to build, are not very effective in reducing hydraulic flows. Unless an effective maintenance program can be devised, or the OBI design modified there is little benefit to installing these controls. Costs and benefits of drain trenches and ditches are compared in Table 3-21 for a 150 ft. underground trench (two 75 ft. trenches connected to one inlet) or surface ditch. Using the costs of Table 3-20, drain trenches would be about four times more costly to build and maintain.

It is difficult to estimate a detention pond size equivalent to a 150 ft. ditch or drain trench. However, a rough cost estimate can be made for excavation, sodding and fencing of the 2.9 acre South Pond. The average depth of this pond is approximately 3 feet and the effective storage capacity is 6.8 acre-feet. Assuming the entire storage area was excavated yields 11,000 cu. yd. of material at a total cost of about $50,000.

TABLE 3-21. COMPARISON OF COSTS AND BENEFITS FOR A 150-FT. DRAIN TRENCH OR DITCH

Stormwater control	Capital costs ($)	O&M costs ($/year)	Expected load removal (%)
Drain trench	3,170	53.40	30 - 35
Ditch	850	15.00	30 - 70

Sodding costs for the pond would be $27,000 and fencing costs for a perimeter of 1,900 ft. would be about $8,700. Capital costs of construction alone would be $86,000. Annual maintenance would be minimal in comparison. The benefits associated with this pond include a potential of high removal of particulate matter present in the stormwater runoff. This removal is highly variable, however, and very dependent on rainfall characteristics, antecedent conditions, groundwater levels, and in-pond available storage.

The relative area served by detention ponds and drain trenches or ditches can be compared. South Pond, for example, receives inflow from about 75 acres which yields approximately 1 acre of detention area to 40 acres of drainage area. From Tables 3-6 and 3-11 the drainage areas served by ditches or drain trenches are about 1 to 3 acres. Assuming conservatively that the drain trench and ditch drainage areas are both 1 acre, 40 drain trenches or ditches would be necessry to handle the load of one pond. Construction of 40 drain trenches would cost approximately $127,000 with a yearly maintenance cost of $2,000, while forty ditches would cost about $34,000 to construct. If land costs, surveying costs and other unaccounted for detention pond costs were less than $40,000 for a 2-acre pond, the pond would be economically preferable to 40 drain trenches. The cost of 40 ditches, however, is less than half of the accountable detention pond costs. Other factors that must be taken into account are that removal capabilities of ponds are less reliable, and that ditches in residential areas are not always desirable.

Overall Assessment of Stormwater Control Performance

Various stormwater controls currently in use in the City of Tampa have a wide range of flow, and subsequently, pollutant removal capacities. The discussion presented above is summarized by the following points:

1. The design of open bottom inlets is not favorable to significant hydraulic removal of stormwater runoff. Although open bottom inlets are inexpensive and easy to construct and maintain, their value is limited by their ineffectiveness under existing maintenance and design practices. Even if OBI'S did work well, their available removal storage is so small that they may not be economically feasible or desirable in a stormwater control plan.
2. The hydraulic and pollutant removal rate of drain trenches ranges from 30 to 35 percent of the inflow rates in dry and wet weather conditions. Drain

trenches are somewhat costly to construct and maintain but their underground location is a desirable feature.

3. Ditches have a long term removal rate of 30 to 70 percent of the inflow rate. Undercut ditches have a long term removal rate of 50 to 70 percent of the inflow rate. Ditches are less expensive to construct and their above-ground location allows for easy maintenance, but their presence in residential areas is not met favorably by the public.
4. Detention ponds reduce the peak rate of inflow from 20 to 95 percent and remove various pollutants to a similar extent. Detention ponds require high capital investment and are limited by land availability. Public opposition to detention ponds can also be significant.

The performance evaluation and costs/benefit analysis that is presented here will be taken into account in development of the Stormwater Management Plan for the City of Tampa.

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2. Priede-Sedwick, Inc., Control Testing Water Quality and Flow Data, Task II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, March, 1983.

3. Priede-Sedwick, Inc., Detailed Event Data, Tasks II.5 and II.8, Tampa Nationwide Urban Runoff Program Study, Phase II, April, 1983.

4. Roy Jorgensen Associates, Inc., Jorgensen Work Management System, Gaithersburg, MD, 1982.

CHAPTER 4

PROJECTION OF STUDY AREA-WIDE LOADINGS

General

The major objective of the Tampa NURP study is to develop a stormwater management plan for the City which takes into account stormwater quantity (flooding problems) as well as its quality (state permitting and receiving water impacts). In order to develop the City's stormwater management plan, it is necessary to quantify the stormwater flows and loads for the entire study area. Specific needs for development of loadings from the entire study area include:

1. Quantification of long-term stormwater runoff flows for design conditions to enable assessment of needed flooding controls.
2. Quantification of long-term stormwater runoff loading (based on flow and concentration) for design conditions to enable stormwater management planning for protection of receiving water quality. This will allow two major objectives to be accomplished:
    a. The stormwater management plan will be consistent with intended uses and desired quality of the receiving water (lower Hillsborough River) as specified in the State water quality standards (1).
    b. The City of Tampa will have a basis with which to statisfy requirements for State of Florida

stormwater permits as described in Chapter 17-25 of the Florida Administrative Code.

3. Quantification of single event stormwater flows and loads for calibration of the lower Hillsborough River model, and projection of stormwater impacts.

In this Chapter, the generation of these study area-wide flows and loads are described.

Approach to Study Area-wide Load Development

In Chapter 2, simulation of individual basins was performed using the detailed single event model SWMM(2). The use of single event models allows the important factors in runoff flow and load generation to be identified. Single event models, however, are not suitable for use with long term rainfall records such as years of low, average and high rainfall because such use would require a considerable amount of computer time and related expenses. Because of this, simplified models such as S/SWMM(3), are used to generate runoff volumes and pollutant loads for long term rainfall records.

The simplified models use runoff and pollutant factors that are either developed for each basin using the single event model (SWMM) or measured in a field study. For the Tampa NURP Project, both techniques were used. The simplified models also generate the statistics (e.g. frequency distribution) of flows and loads discharged into the receiving waters on a seasonal and yearly basis. The simplified models are essentially bookkeeping techniques similar to use of the Rational Method. For this study, the S/SWMM model developed by Metcalf & Eddy will be used to generate area-wide flows and loads.

Like the SWMM model, the input data which drives the S/SWMM model consists of rainfall records. In order to effectively manage the long term records, the processes which translate the rainfall into runoff and potential overflows have been simplified compared with those of the SWMM model.

Given long term rainfall records, S/SWMM calculates the quantity and quality of runoff, the effect in-system storage may have on the amount of runoff and the volume and pollution load of storm flooodwater. This was done for each subarea within the study area on an hourly basis for representative wet, average and dry years previously selected from the overall available rainfall record (4). The S/SWMM model is illustrated in Figure 4-1. Further comparisons of SWMM and S/SWMM were presented in Chapter 2.

S/SWMM Model Inputs and Outputs

Development of the long-term flows and loads using S/SWMM requires that the rainfall record for the period of study be specified and that the overall drainage area be discretized into subareas. Further, the subareas must be assigned input values for the following:

1. Runoff coefficient
2. Storage volume
3. Outflow capacity
4. Pollutant concentration

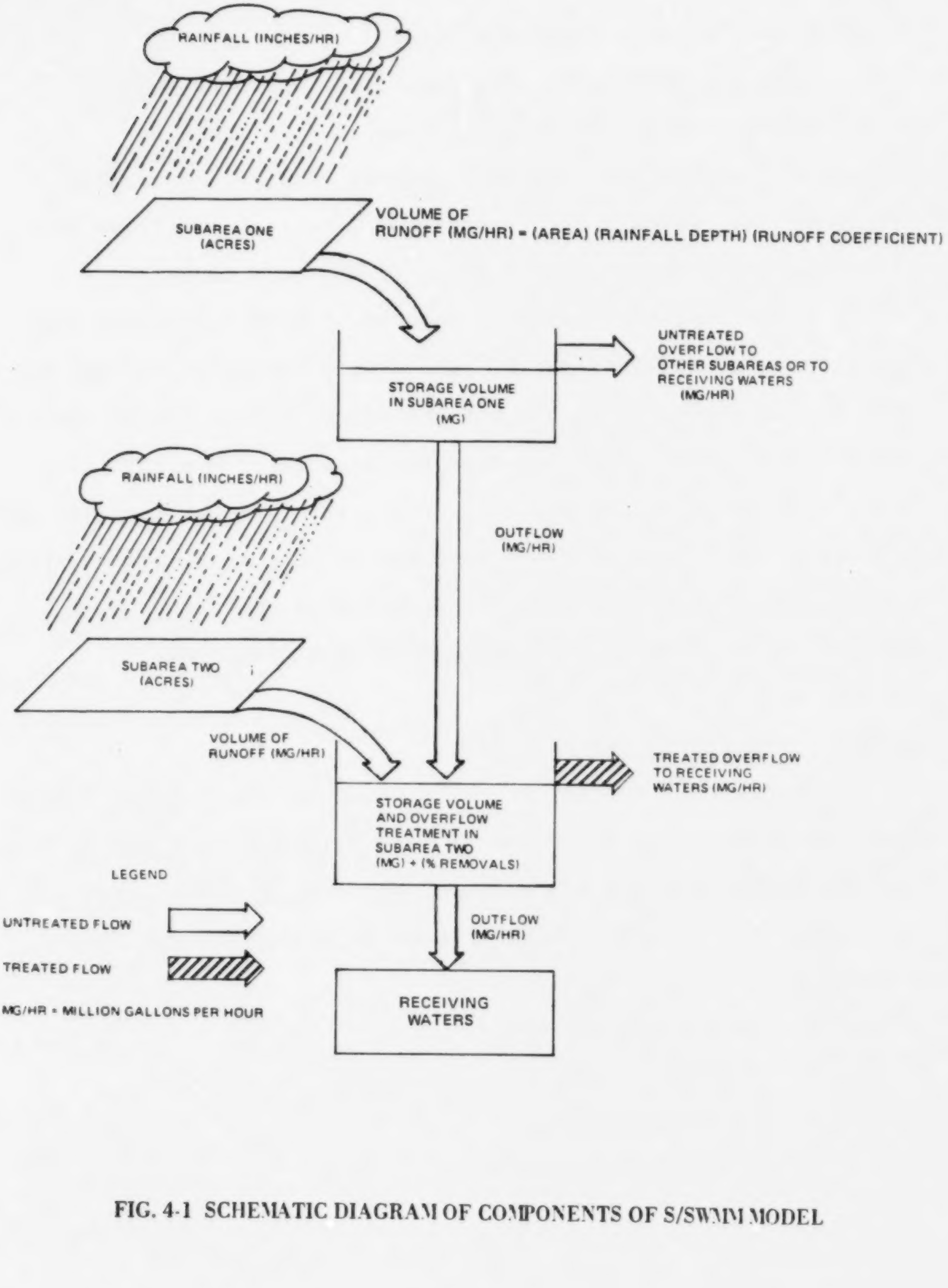
RAINFALL (INCHES/HR)
SUBAREA ONE (ACRES)
VOLUME OF RUNOFF (MG/HR) = (AREA) (RAINFALL DEPTH) (RUNOFF COEFFICIENT)
STORAGE VOLUME IN SUBAREA ONE (MG)
UNTREATED OVERFLOW TO OTHER SUBAREAS OR TO RECEIVING WATERS (MG/HR)
RAINFALL (INCHES/HR)
OUTFLOW (MG/HR)
SUBAREA TWO (ACRES)
VOLUME OF RUNOFF (MG/HR)
TREATED OVERFLOW TO RECEIVING WATERS (MG/HR)
STORAGE VOLUME AND OVERFLOW TREATMENT IN SUBAREA TWO (MG) + (% REMOVALS)
LEGEND
UNTREATED FLOW
TREATED FLOW
MG/HR = MILLION GALLONS PER HOUR
OUTFLOW (MG/HR)
RECEIVING WATERS

FIG. 4-1 SCHEMATIC DIAGRAM OF COMPONENTS OF S/SWMM MODEL

Given these inputs, S/SWMM will predict flooding occurrence and volume, and pollutant load to the receiving waters. The following is a description of each component of S/SWMM and of how the input values were generated.

Rainfall. Hourly rainfall as recorded at the Tampa International Airport U.S. Weather Bureau rain gage was input to the model in inches per hour. The entire available rainfall record from 1949 to 1979 inclusive was analyzed (4) to determine representative wet, average and dry years for S/SWMM simulation. The wet, average and dry years selected were 1969, 1971 and 1974 respectively.

The average year was selected based on the mean annual rainfall and the mean annual number of storms from the available years of record. The wet and dry years were chosen to closely approximate annual rainfall and number of storms one standard deviation above and below these mean values. The characteristics of these years are summarized in Table 4-1. A storm event was defined as recorded rainfall preceded by or separated from other recorded rainfall by at least six hours during which no measurable rainfall was recorded.

The three years were selected to be statistically representative of the entire period of record in order to develop frequency distributions of overflows without simulating the entire period of record.

Drainage Area Discretization. The surface area that drains each subarea was input to the model in acres. The area of

TABLE 4-1. CHARACTERICTICS OF REPRESENTATIVE AVERAGE, WET AND DRY YEARS

Year	Annual rainfall (in.)		No. of Storms
Overall record	46.10(1)	(11.53)(2)	100.5(1)(10.7)(2)
Average year (1971)	46.33		95
Dry year (1974)	33.90		78
Wet year (1969)	54.22		110

1. Mean value.
2. Standard deviation.

study, the Lower Hillsborough River Basin, was represented in S/SWMM as 17 distinct subareas, each outflowing and overflowing directly to the river. City of Tampa Map C, entitled "Storm Drainage," which indicates major and minor drainage subareas in the City was used to delineate the subarea boundaries. Using City of Tampa nomenclature, the subareas input to S/SWMM correspond to "minor drainage basins" of the lower Hillsborough River basin, which is a "major drainage basin." The subareas as input to the S/SWMM model are shown in Figure 4-2. Pertinent subarea characteristics are presented in Table 4-2. These are described in more detail below. The subareas range in size from 469 to 1524 acres, with an average size of about 860 acres.

Runoff Coefficient (Rc). A runoff coefficient is used to determine the percentage of rainfall volume in each subarea that is converted to runoff. Subarea runoff coefficients were determined as a function of projected land use as given in

TABLE 4-2. S/SWMM SUBAREA CHARACTERISTICS FOR THE EXISTING STORMWATER SYSTEM WITH PROJECTED LAND USE

Subarea						
No.	Name	Abbreviation	Runoff Area (acres)	Runoff Coefficient	Storage (mgal)	Outflow (mgal/hr)
1.	North Tampa	NT	929	.45	13.22	1.00
2.	Seaboard Coastline	SC	811	.56	8.32	2.00
3.	Sulphur Springs	SS	502	.47	.77	10.70
4.	River Bend	RB	873	.43	.62	27.43
5.	Forest Hills	FH	1524	.51	14.00	5.87
6.	Seminole Heights	SH	575	.43	.46	7.55
7.	Northwest Tampa	NW	1160	.39	31.44	.99
8.	Kirby Creek	KC	1131	.41	6.17	14.12
9.	Oak Grove	OG	469	.38	.95	20.25
10.	Hillsborough Ave.	HA	992	.46	3.54	5.97
11.	Wellswood	WW	666	.48	.55	18.19
12.	Sunshine Park	SP	856	.42	.48	17.96
13.	St. Joseph's	SJ	1022	.47	13.08	13.94
14.	Tampa Heights	TH	1071	.44	5.30	4.60
15.	West Tampa	WT	898	.43	.82	9.24
16.	University of Tampa	UT	657	.59	.58	33.00
17.	Downtown	DT	525	.63	.57	39.59

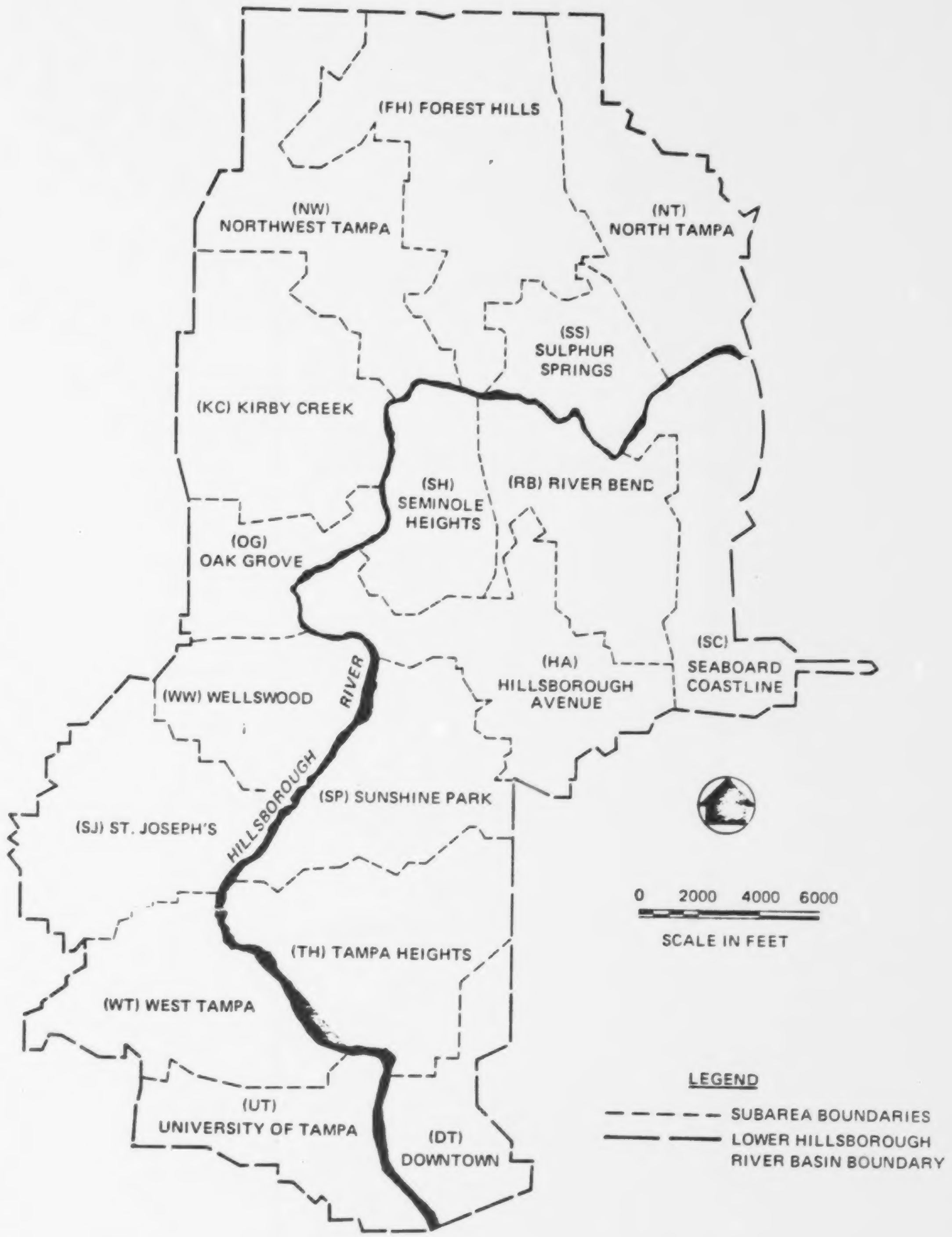
(FH) FOREST HILLS
(NW) NORTHWEST TAMPA
(NT) NORTH TAMPA
(SS) SULPHUR SPRINGS
(KC) KIRBY CREEK
(SH) SEMINOLE HEIGHTS
(RB) RIVER BEND
(OG) OAK GROVE
(SC) SEABOARD COASTLINE
(HA) HILLSBOROUGH AVENUE
(WW) WELLSWOOD
HILLSBOROUGH RIVER
(SP) SUNSHINE PARK
(SJ) ST. JOSEPH'S
0 2000 4000 6000
SCALE IN FEET
(TH) TAMPA HEIGHTS
(WT) WEST TAMPA
(UT) UNIVERSITY OF TAMPA
(DT) DOWNTOWN
LEGEND
SUBAREA BOUNDARIES
LOWER HILLSBOROUGH RIVER BASIN BOUNDARY

FIG. 4-2 LOWER HILLSBOROUGH RIVER BASIN DRAINAGE SUBAREA DISCRETIZATION

City of Tampa Preliminary Subsector Plans, Tampa 2000 Land Use Element. A runoff coefficient for each land use was assigned based on analysis of the detailed storm event and percent imperviousness data presented in Chapter 2.

Two methods were used to calculate runoff coefficients. The first involved determining the volumetric coefficient based on the measured data as follows:

$$\text{Rc (measured)} = \frac{\text{runoff volume}}{\text{rainfall volume}}$$

As described in Chapter 2, Rc (measured) was calculated using all reasonable data from the storm events in which rainfall and runoff measurements were available. These coefficients were then averaged to obtain basin-aggregate, measured Rcs, which are presented in Table 2-9 and again in Table 4-3.

The second method involved estimation of Rc based on the percent imperviousness of each basin which is given in Table 2-9. This was shown in Chapter 2 to be an important factor affecting hydraulic simulations. Using a coefficient of 0.15 for pervious areas and 0.95 for impervious areas, the following relationship was used for estimation:

$$\text{Rc (predicted)} = \frac{\text{Rc}_p \text{ (\% Pervious)} + \text{Rc}_{imp} \text{ (\% Impervious)}}{100}$$

where

Rc_p = runoff coefficient for pervious areas, 0.15

Rc_{imp} = runoff coefficient for impervious areas, 0.95

Table 4-3 compares the values of the measured and predicted Rc for each basin.

In all basins except Norma Park the measured and predicted values correlate well. The measured runoff coefficient was chosen as the representative Rc for these basins since it reflects average, real event response.

In the Norma Park basin, measured runoff is significantly less than predicted runoff. This is attributed to the undercut ditch system in this basin. As demonstrated in both Chapters 2 and 3, these ditch systems are highly effective in reducing runoff. Since these undercut ditch systems are not widespread in the study area, the predicted value of Rc was used to obtain runoff coefficients for commercial land use.

From knowledge of the land use distribution in the monitored basins as specified in Table 2-9, a characteristic Rc for each land use was determined. The J.L. Young basin is 100 percent high density residential. Thus the Rc for high density residential corresponds to the Rc measured for this basin, which is 0.68. If an Rc of 0.15 is assumed for open/natural areas, then an Rc of 0.37 can be derived for low density residential areas from Charter & Harding, which is a mix of these two land uses. A medium density residential Rc of 0.53 was assigned as the average of the high and low density residential Rcs. Knowing the medium density residential Rc, leads to the determination of an industrial/commercial Rc from Norma Park, which is a mix of these two land uses. The derived Rc values are given in Table 4-4.

TABLE 4-3. MEASURED AND PREDICTED RUNOFF COEFFICIENTS FOR THE FIVE BASINS

Basin	R_c (measured)	R_c (predicted)
J.L. Young	.68	.64
Wilder Ditch	.45	.46
North Jesuit	.34	.45
Charter and Harding	.35	.40
Norma Park	.66	.87

TABLE 4-4. RUNOFF COEFFICIENTS DERIVED FOR VARIOUS LAND USES

Land use	Runoff coefficient
Industrial/Commercial	0.90
High Density Residential	0.68
Medium Density Residential	0.53
Low Density Residential	0.37
Institutional	0.31
Open/Natural	0.15

The Subsector Land Use Maps contained 15 land uses which were grouped with respect to the 6 land uses shown in Table 4-4. An Rc of 0.90 was used for all forms of commercial and industrial land use. An Rc of 0.31 was used for public and semi-public areas. Based on these coefficients, the subarea runoff cofficients as listed in Table 4-2 were calculated. The values range from 38 percent in less developed areas to 63 percent in central urban areas. The runoff coefficient for each area is assumed to remain constant over time.

Storage Volume. Each subarea has potential for storage that may alter the amount and timing of runoff leaving the subarea. This storage consists of the volume in pipes, ditches and retention ponds which must be filled before an overflow can occur. An estimation of pipe and ditch length in each subarea was made based on Map C City Storm Drainage. An average pipe diameter for one representative subarea (Sunshine Park) of 48 inches was determined from detailed maps and was used to calculate pipe storage volume in all other subareas. The ditch cross section used was 48 sq.ft. which is a cross-sectional area typical of Tampa major ditches as specified by the City. Retention pond surface area was determined from Map E, Stormwater Retention Areas in the City of Tampa. A typical average storage depth of 3 feet was used for these ponds. Storage was input in each subarea as volume in million gallons.

Outflow Capacity. The outflow rate is the rate at which runoff is discharged through the drainage network. Outflow volume can be routed into another subarea or directly to the receiving water. In the lower Hillsborough River basin subareas, all major outflows lead directly to the river. The number and size of the outfalls from each subarea with a 24 to 36 inch capacity was determined from the City of Tampa map entitled Major Outfalls. A pipe slope of 0.004 was assumed for these outfalls, which contribute only a small percentage of overall capacity. For outfalls with greater than a 36 inch capacity, the City provided detailed information, which is summarized in Table 4-5a for pipes and 4-5b for concrete boxes. This enabled determination of subarea outflow capacities. As the crowns of all the outfalls that detailed information was available on were above MSL, Manning's equation for pipes flowing full was used to compute specific outfall capacities:

$$Q = \frac{1.49\,A}{n} R^{2/3} S^{1/2}$$

where

Q = flow rate, ft^3/sec

R = hydraulic radius = $\frac{\text{cross-sectional area of flow}}{\text{wetted perimeter}}$

S = slope of energy grade line (pipe slope for unsubmerged outfalls)

n = roughness coefficient = 0.015 (typical of concrete pipe in fair condition)

Outflow capacities are input to S/SWMM in million gallons per hour. If the subarea outflow capacity is exceeded the volume either fills available storage, or floods.

Flooding. Flooding occurs in a given subarea when, in any given hour, the storage capacity and outflow rate are exceeded. Floodwaters in the lower Hillsborough River basin subareas are all routed directly to the river. The rate of flooding in million gallons per hour is calculated by the model and depends on the rainfall rate, the subarea characteristics and the amount of runoff.

It should be noted that S/SWMM will not predict loaclized flooding due to insufficient capacity upstream of the subarea outfalls. Only when outfall capacity is limiting will flooding occur.

TABLE 4-5. CHARACTERISTICS OF OUTFALLS LARGER THAN 36 INCHES

a. PIPES

Subarea abbrev.	Diameter (inches)	Slope (ft/100 ft)	Invert El. from MSL (feet)
SS	60	0.71	4.8
SS	60	0.35	-2.6
RB	42	1.20	-1.0
RB	42	2.30	2.8
RB	72	1.00	-3.5
RB	72	1.00	-3.5
FH	78	0.23	-3.5
KC	72x44(1)	1.50	-0.7
OG	48	0.36	0.1
OG	66	1.00	1.0
OG	76x48(1)	1.16	2.0
SP	60	0.64	-3.3
SP	54	0.40	-0.6
SP	54	0.40	-0.6
SP	48	1.00	-0.1
SJ	42	1.73	2.3
SJ	60	1.00	0.0
SJ	60	1.00	0.0
TH	48	.61	-2.9
WT	66	.45	-2.0
WT	65x40(1)	0.19	1.4
UT	48	0.47	-3.2
DT	72	0.30	-3.1
DT	48	2.50	-0.8

b. CONCRETE BOXES

Subarea abbrev.	Dimensions (feet)	Slope (ft/100 ft)	Invert El. from MSL (feet)
SH	8.0x3.9	0.12	-1.3
NW	6.0x6.0	0.01	0.0
KC	10.0x6.0	0.20	-2.0
OG	9.0x3.0x2.0(2)	2.00	-0.1
HA	7.0x4.5	0.47	1.5
WW	3.0x3.0	0.82	1.3
WW	14.0x6.5	0.32	-4.1
UT	12.0x4.0	1.40	-0.4
UT	4.0x8.0	1.40	-0.4
UT	3.0x5.0	NA(3)	-1.0
UT	3.0x5.0	NA	-1.0
DT	13.0x6.0	0.10	-4.9
DT	6.0x4.0	0.30	-3.1
DT	4.0x4.0	NA	NA
DT	6.5x8.0	0.81	-1.7

---

1. Oval shaped.
2. Trapezoidal shaped.
3. Not available.

Pollutants. The concentration of pollutants is input to the model as a constant in units of milligrams per liter for all pollutants except total coliform, which is in units of Most-Probable-Number per 100 milliliters (MPN/100 ml). Since S/SWMM pollutant concentrations are constant for the duration of the storm, they should be derived from the flow weighted, composite data, as opposed to the discrete data, which reflects within storm variability. The three pollutants selected for study were total suspended solids (TSS), five-day biochemical oxygen demand (BOD_5), and fecal coliforms. Those stormwater constituents were identified as potential problem pollutants in the lower Hillsborough River during Phase I (5) of the NURP study. In Chapter 2, it was determined that no significant correlations existed between pollutants and either composite storm parameters such as antecedent condition, storm duration and intensity, rainfall volume, and runoff flow or land use. Thus, the S/SWMM pollutant concentrations should be independent of these parameters. Therefore, pollutant concentrations that were representative of the stormwater from the entire lower Hillsborough River basin were used in the model. These concentrations were ascertained as follows.

In order to calculate representative values of stormwater concentrations and expected variation about the representative values, the composite data for each pollutant was analyzed. A determination of the probability distributions of the data was made, and the mean and standard deviation of these distributions was calculated. This method of calculation will allow model simulations to account for typical loads generated, as well as the expected variation of these loads.

It was found that BOD_5 and fecal coliform, flow weighted, composite concentrations were normally distributed. Normality was demonstrated by plotting the frequency distribution of the data on probability paper. If the data are from a normal (Gaussian) distribution, the frequency distribution will be linear. Figures 4-3 and 4-4 illustrate the linearity of this plot for BOD_5 and fecal coliforms. It was found that TSS was lognormally distributed. Figure 4-5 shows that the probability plot for the logarithms of the TSS data is reasonably linear, indicating that the TSS data are lognormally distributed. (Fecal coliform data was collected only during discretely sampled events. Thus fewer samples of coliform are available for analysis than for BOD_5 and TSS.)

The median of both normal and lognormal distributions is that value which has the highest probability of occurring. This value for the TSS, BOD_5, and fecal coliform distributions was used in S/SWMM simulation. Due to the high variability of the quality data as discussed, S/SWMM was also run with quality concentrations representative of a prescribed confidence interval around the median values. The prescribed confidence interval is one standard deviation, or about 33 percent on the probability plot. These values are listed in Table 4-5. The high and low values for TSS are unsymmetrical due to the nature of the lognormal distribution.

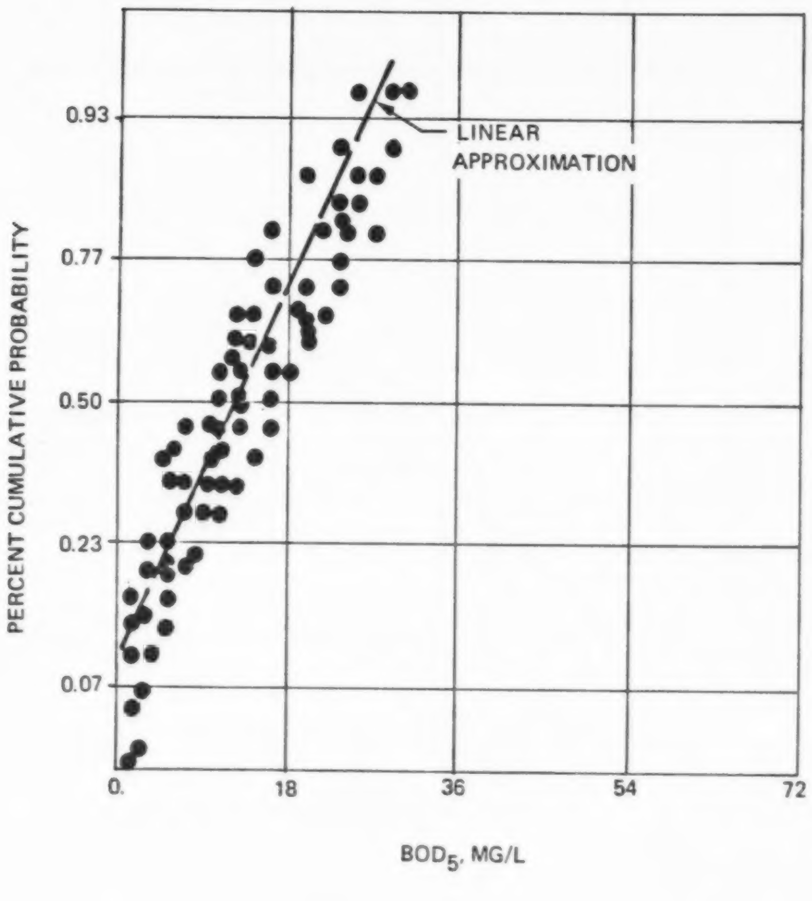

FIG. 4-3 CUMULATIVE PROBABILITY PLOT OF BIOCHEMICAL OXYGEN DEMAND DATA

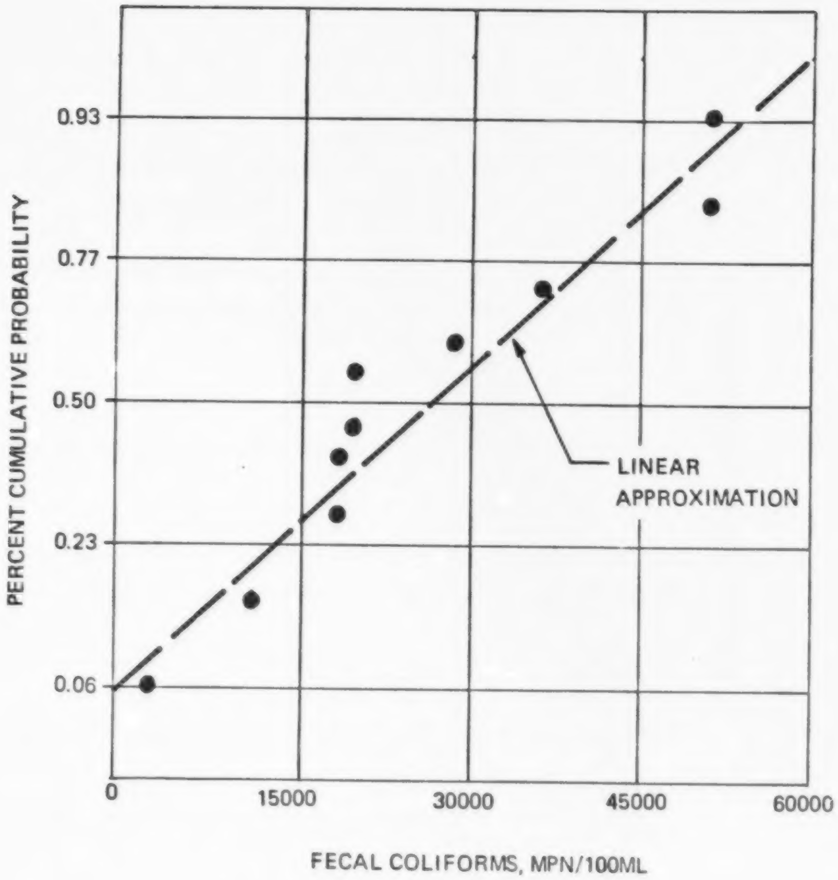

FIG. 4-4 CUMULATIVE PROBABILITY PLOT OF FECAL COLIFORM DATA

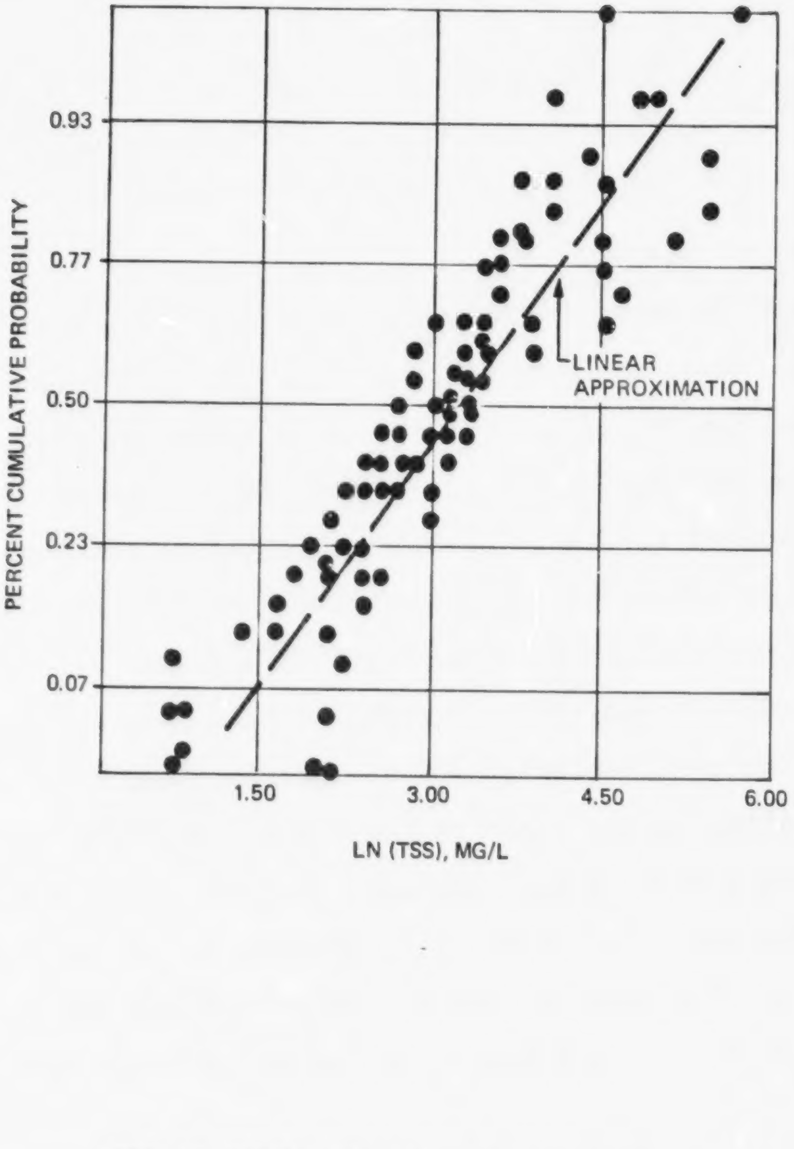

FIG. 4-5 CUMULATIVE PROBABILITY PLOT
OF LOGARITHMS OF TOTAL SUSPENDED SOLIDS DATA

TABLE 4-6. RANGE OF QUALITY CONCENTRATIONS TO BE SIMULATED IN S/SWMM

	Minus one standard deviation	Median	Plus one standard deviation
TSS (mg/l)	7.9	22.2	62.4
BOD_5 (mg/l)	3.8	14.1	24.4
Fecal Coliforms (MPN/100 ml)	9,030	25,400	41,800

S/SWMM Model Check

Before using S/SWMM to generate the flows and loads for the study area, the model simulations were checked using the field measurement data. For each of the five basins, the sum of the runoff volume and the sum of pollutant loads of TSS and BOD_5 was calculated for the composite storm events for which all three values were available. The continuous rainfall records for each of these composite events was input to S/SWMM. The value of Rc (measured) for each basin as given in Table 4-3 was used. Simulations were performed for the median, high and low quality concentration values given in Table 4-5. Predicted total flow volume and pollutant mass was then compared to the measured data.

The results of the S/SWMM simulations are presented in Table 4-6. The observed and predicted outflows correspond within a range of 61 to 122 percent. For each basin and each pollutant,

TABLE 4-7. CHECK OF S/SWMM SIMULATIONS AGAINST COMPOSITE STORM EVENT DATA

Basin	No. of events in comparison	Outflow (mgal)		TSS load (lbs)				BOD_5 load (lbs)			
		observed	predicted	observed	predicted[1]			observed	predicted[1]		
					low	median	high		low	median	high
J. L. Young	14	2.0	2.4	640	160	440	1,230	180	75	280	480
Wilder Ditch	17	36.4	43.0	10,300	2,400	6,700	19,000	5,600	1,150	4,300	7,400
North Jesuit	19	6.2	3.8	1,800	250	710	1,980	640	120	450	780
Charter & Harding	15	9.6	8.8	2,200	580	1,640	4,600	900	280	1,040	1,800
Norma Park	8	3.6	4.4	700	290	810	2,280	470	140	515	890

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1. The quality loads are calculated for the median value as well as one standard deviation above and below it.

the observed total loads fall within the predicted total load range.

It should be noted that the simulation of stormwater flows and loads on an area-wide basis relies on the assumption that the measurements conducted at specific locations can be used to characterize the entire study area. This has been demonstrated for hydraulic as well as water quality data. The following facts support the assumption of transferability of data within the study area:

1. Flow data at monitored basins have been shown to be a predictable function of impervious area.
2. Basin to basin variation in stormwater quality has been shown to be statistically uncorrelated to storm or basin parameters. Therefore this variability is expected to be representative of variability for the entire study area, regardless of storm and basin conditions. It should be noted, however, that due to the significant variability associated with stormwater quality parameters, large data sets are required to capture significant correlation. It is possible that given a larger data set in this study, statistical correlations would have been captured.
3. The S/SWMM model with measured runoff coefficients and predicted quality was checked against measurements and was found to perform well.

Analysis of S/SWMM Model Predictions

Simulations were made for the representative wet (1969), average (1971) and dry (1974) years. The hydrologic simulations allow an assessment of the extent and causes of flooding. Each year was run with three sets of quality concentrations which represent the median and variation of the stormwater quality. The quality simulations allow problem areas to be identified in terms of pollutant loads. Thus, the impact to the receiving waters can be addressed, and needed controls identified during the development of the City's Stormwater Management Plan.

Quantity Predictions. Table 4-8 characterizes flooding in the lower Hillsborough River basin for the three years of simulation. The flood volume/runoff volume and the flooding storms/total storms ratios are measures of the performance of the stormwater system as the hydraulic load on it increases. The values of these ratios increase from the dry year to the average year and stay approximately constant into the wet year. Flooding volume increases with increasing hydraulic load. The rate of this increase, however, diminishes as annual load increases. Note that the average year flooding volume is three times that of the dry year, while wet year flooding volume is only 1.2 times that of the average year.

A review of the model results indicated that storms with at least one hourly intensity greater than 1 inch per hour were the major cause of flooding. Total rainfall volume was less of a determining factor. For the wet year, there were seven hourly intensities greater than 1 inch per hour. These were responsible

TABLE 4-8. HYDROLOGIC SUMMARY FOR LOWER HILLBOROUGH RIVER BASIN IN WET, AVERAGE AND DRY YEARS

	Wet	Average	Dry
Flooding volume (MG)	364	319	102
Runoff volume (MG)	10,100	8,660	6,340
$\frac{\text{Flood Volume}}{\text{Runoff Volume}}$ x100(%)	3.6	3.7	1.6
Sum of subarea flood events	56	42	23
Total number of events	110	95	78
$\frac{\text{Flooding events}}{\text{Total number of events}}$ x100(%)	13	11	9
Total annual rainfall (inch)	59.2	46.3	33.9

for about 90 percent of the total flooding volume for most of the subareas. For the average year, there were five hourly intensities greater than 1 inch per hour. Two of these intensities occurred on consecutive hours during the event of September 9, 1971. This storm resulted in the worst flooding in the entire three years of record for most subareas. Flooding during the dry year was relatively limited. Only one event had an hourly intensity greater than 1 inch per hour, and this caused most of the flooding.

A breakdown of flooding volume by subarea and simulation year is given in Figure 4-6. Eight of the seventeen basin subareas flood. Three of the subareas, West Tampa, Seminole Heights and Northwest Tampa, flood more during the average year

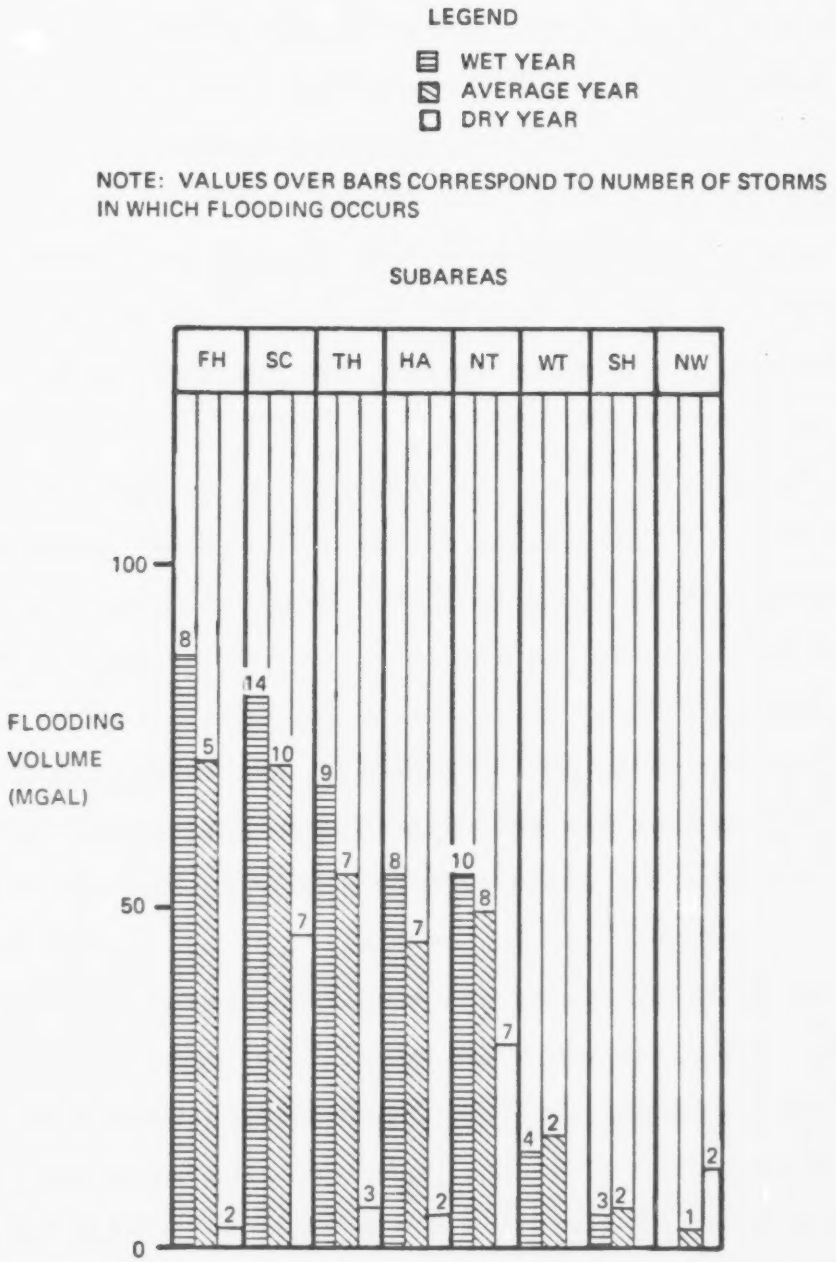

FIG. 4-6 FLOODING SUMMARY BY SUBAREA FOR WET, AVERAGE AND DRY YEARS

than the wet year. This is because the average year contains the most intense storm event during the two years, and all three of these subareas flood during this event, which occurred September 9, 1971. Northwest Tampa, having the largest storage volume of any surbarea, will only flood during either an unusually intense sotrm or an unusually large storm. The largest storm event of the three year record occurred on June 26, 27 of the dry year. Thus dry year flooding is larger then wet and average year flooding in Northwest Tampa.

The largest amount of flooding, in decreasing order, occurs in the Forest Hills, Seaboard Coastline, Tampa Heights and Hillsborough Avenue subareas. Drainage from Forest Hills is limited by capacity, however the City has installed pumping capabilities in order to alleviate flooding. The Seaboard Coastline subarea is not residential, and is therefore generally not a problem area for the City. Tampa Heights and Hillsborough Avenue are subareas where current flooding problems exist and complaints are received by the City.

To eliminate flooding, either the storage volume or capacity of the subarea would need to be increased to a level sufficient to handle all flow from a design storm event. In-system storage can be increased by addition of retention/detention ponds, ditches, or drain trenches. Capacity can be increased by design of a larger drainage system or by installation of pumping capacity. The optimum methods to deal with flooding in a specific subarea will be a trade-off considering costs, receiving water impacts and engineering feasibility. The applicable methods will be identified and presented as part of the City of Tampa Stormwater Management Plan.

Quality Predictions. Table 4-9 summarizes the probable range of pollutant loading to the receiving waters predicted for the three years of simulation. These loads are equal to the total runoff volume in a particular year multiplied by a given loading can be accomplished by using controls such as those discussed in Chapter 3.

Table 4-9 indicates that loading increases with rainfall and concentration, with the worst case being the wet year, high range of concentrations and the best case being the dry year, low range of concentrations. The loading increases approximately an order of magnitude from the best to the worst case.

Figure 4-7 through 4-9 show a breakdown of pollutant loading by subarea. Subarea loading decreases with decreasing (Rc x Runoff Area). Loads are largest in Forest Hills, which has at least 30 percent more acreage than any other subarea, and smallest in Oak Grove, which has the smallest Rc.

It should be noted that subareas with the worst flooding problems are not necessarily those which generate the most pollutant loadings. For example, St. Joseph's has a relatively minor amount of flooding because of its available capacity and in-system storage. However, because of the size of this subarea, it generates the second largest amount of pollution load.

TABLE 4-9. POLLUTANT RUNOFF TO LOWER HILLSBOROUGH RIVER IN WET, AVERAGE AND DRY YEARS

	BOD (lbs x 10^4)			TSS (lbs x 10^4)			Fecal coliform (organisms x 10^{14})		
	H[1]	M	L	H	M	L	H	M	L
Wet	206	119	32	528	188	67	160	97	35
Average	176	102	27	451	160	57	137	83	30
Dry	129	75	20	330	117	42	100	61	22

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1. H = High range, one standard deviation above median, M= median, L = low range, one standard deviation below median.

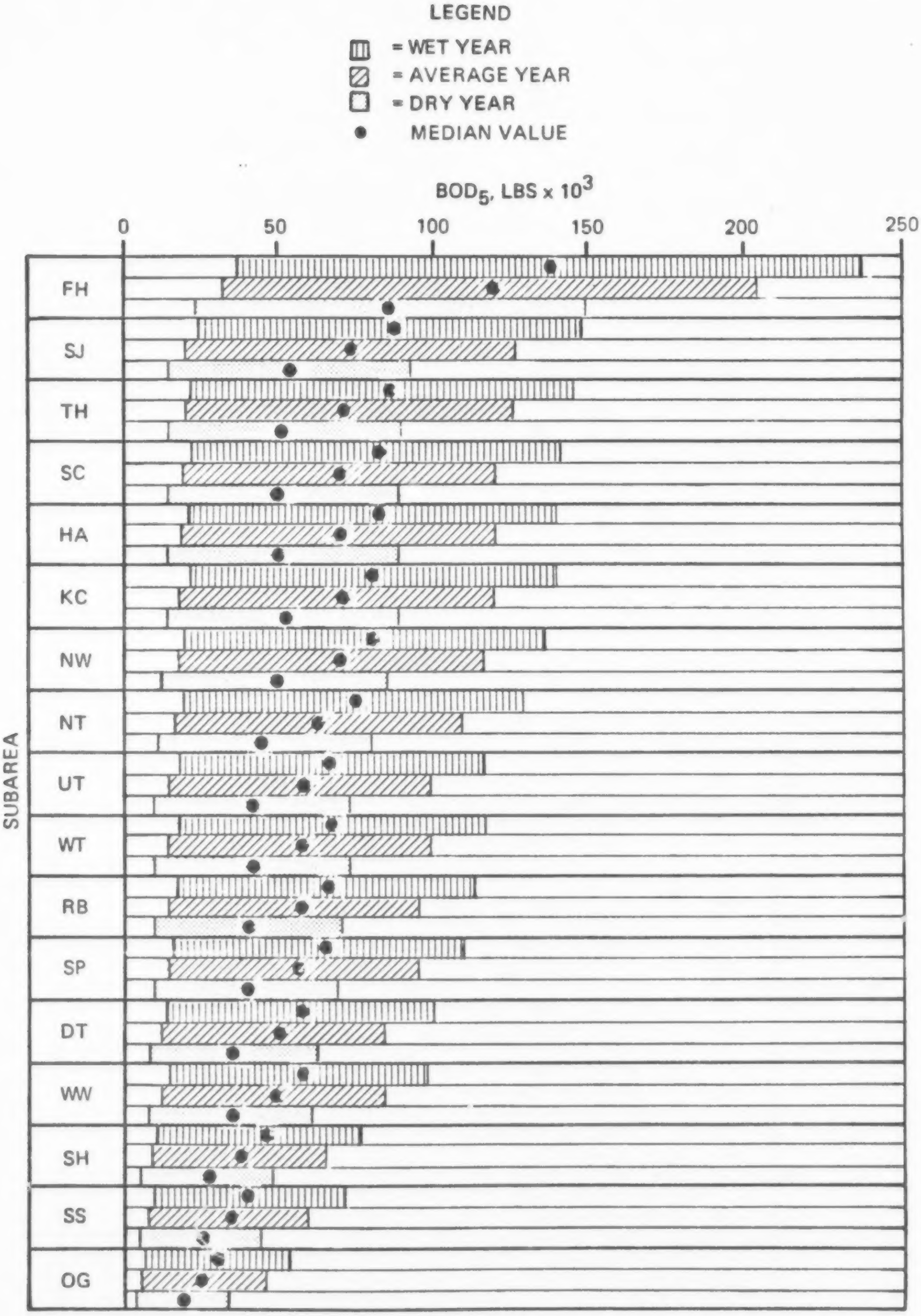

FIG. 4-7 EXPECTED RANGE OF BOD_5 BY SUBAREA
FOR REPRESENTATIVE WET, AVERAGE AND DRY YEARS

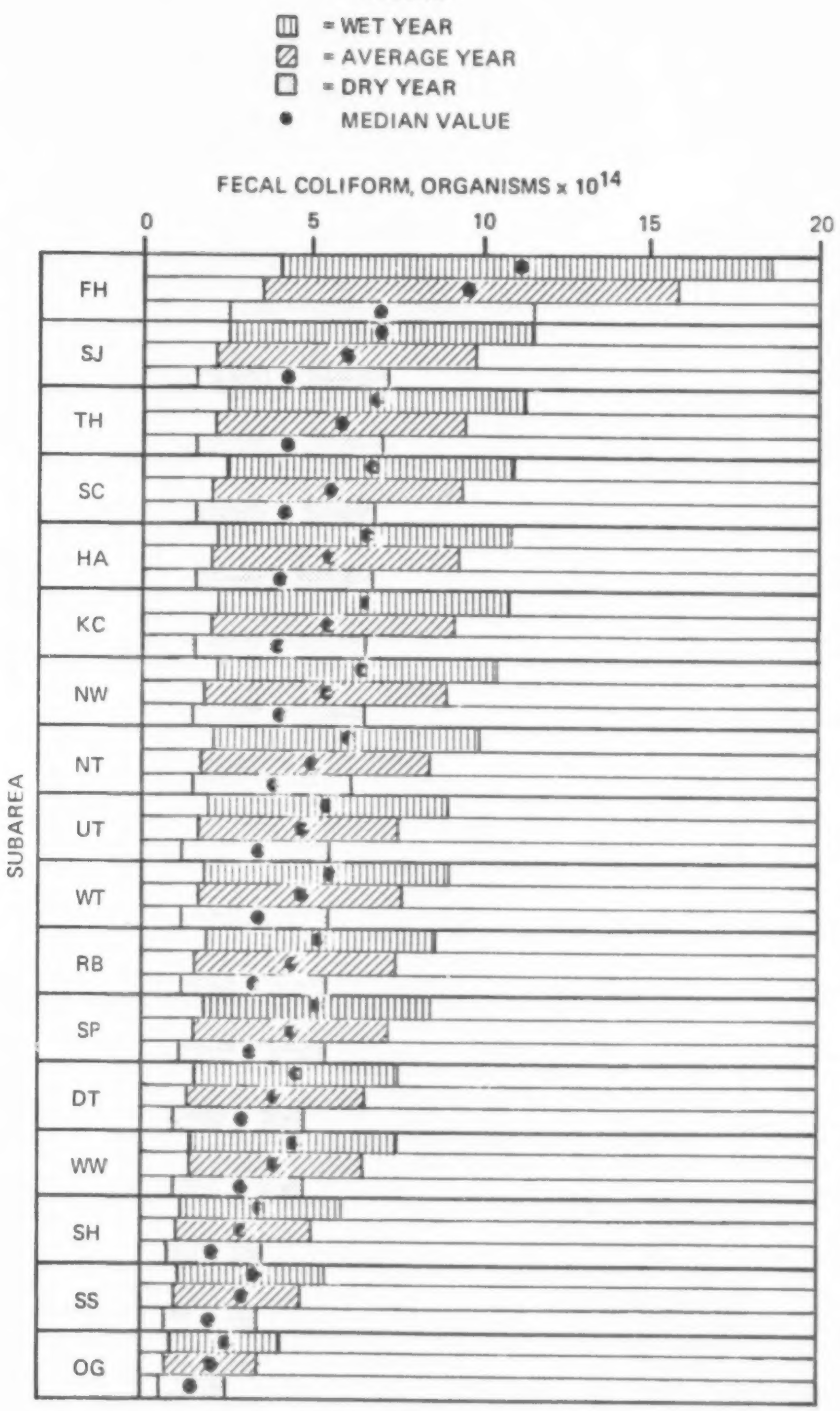

FIG. 4-8 EXPECTED RANGE OF FECAL COLIFORM BY SUBAREA FOR REPRESENTATIVE WET, AVERAGE AND DRY YEARS

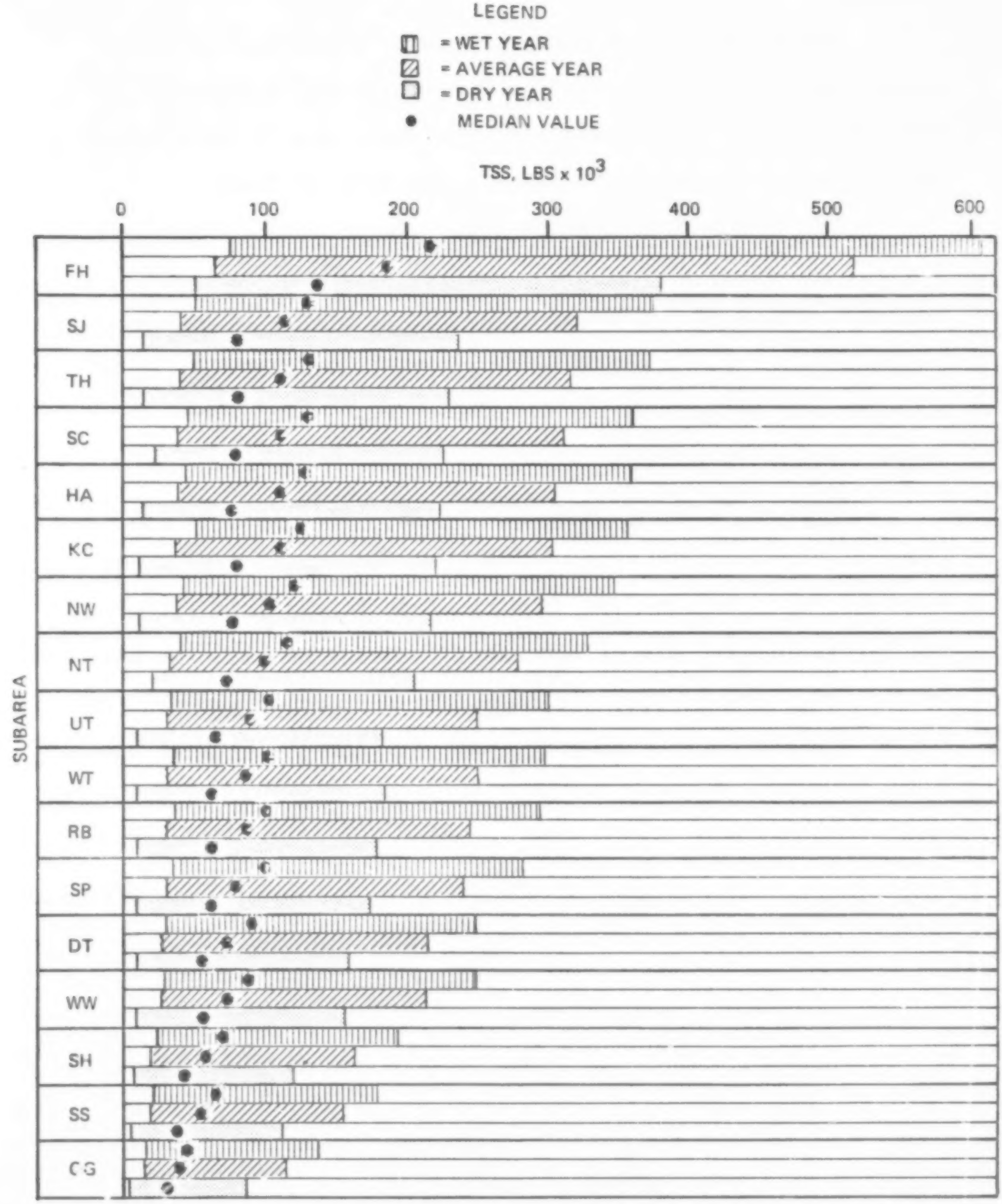

FIG. 4-9 EXPECTED RANGE OF TSS BY SUBAREA FOR REPRESENTATIVE WET, AVERAGE AND DRY YEARS

Summary

The projection of study area-wide flows and loads was accomplished in this Chapter. These flows and loads will be utilized to assess receiving water impacts and in development of the Stormwater Management Plan for the City of Tampa.

The S/SWMM model was used for projection of flows and loads. The study area was discretized based on the minor drainage basins of the City. Model input values were developed from analysis of stormwater controls in Chapter 3. The model was checked against data collected in the five stormwater characterization basins.

Results of the S/SWMM simulations indicated the critical flood-prone areas in Tampa, as well as the importance of rainfall intensity on flooding. To develop plans to mitigate flooding, trade-offs between use of additional storage or capacity must be considered. The pollutant simulations indicated the range of annual loadings to be expected, and identified potential problem areas.